CN rail research

recherche CN rail

CA1
DTC 845
85
E85
c.2

CN

TP 7215E 184

EVALUATION OF MECHANICAL PROPERTIES OF THREE CANADIAN SOILS FOR USE AS RAILROAD TRACK SUBGRADE MATERIAL

Prepared for

TRANSPORT CANADA
TRANSPORTATION DEVELOPMENT CENTRE

Professor R.N. Yong, Director
Dr. P. Boonsinsuk
Mr. F. Caporuscio
Geotechnical Research Centre
McGill University,
Montreal, Quebec

JUL 20 1987

CN Technical Research Centre
3950 Hickmore Avenue
St. Laurent, Quebec
H4T 1K2

February 1985

The contents of this report reflect the views of the contractor and not necessarily the official views or policies of the Transportation Development Centre (TDC) of Transport Canada.

Un sommaire en français de ce rapport est inclus avant la table des matières.

REPORT DOCUMENTATION FORM

1. Transport Canada Report No.	2. TDC Project No.	3. Recipient's Catalogue No.
TP 7215E	4567	

4. Title and Subtitle	5. Report Date
EVALUATION OF MECHANICAL PROPERTIES OF THREE CANADIAN SOILS FOR USE AS RAILROAD SUBGRADE MATERIAL	February 1985
	6. Performing Organization Report No. 184

7. Author(s)	8. Transport Canada File No.
Professor R.N. Yong, Dr. P. Boonsinsuk, Mr. F. Caporuscio	D1465-108-6-9

9. Performing Organization Name and Address	10. DSS File No.
Geotechnical Research Centre McGill University Montreal, Quebec	11SD.T8200-1-1572
	11. DSS or Transport Canada Contract No. OSD81-00186

12. Sponsoring Agency Name and Address	13. Type of Report and Period Covered
Transportation Development Centre (TDC) Guy Favreau Complex 200 Dorchester Blvd. W., Suite 601, West Tower Montreal, Quebec H2Z 1X4	1979-1984
	14. Sponsoring Agency Code TDC 4567

15. Supplementary Notes	16. TDC Project Officer
Canadian National Railways and TDC equally shared the cost of this program.	D.W. Dibble

17. Abstract

Work on this project aimed at predicting granular (ballast and subballast) depth requirements for railroad track was initiated by Canadian National Railways Technical Research Centre in 1979. The Geotechnical Research Centre of McGill University was retained for laboratory testing of three soil types, two of which were Lake Edmonton clays, with the third one being a sandy silt with little clay from Joliette, Quebec.

This is a data report presenting all the laboratory test results obtained in an effort to understand soil behaviour under conditions simulating dynamic train loadings. Basic soil properties, mineralogy, static consolidated - undrained triaxial test results, cyclic triaxial test results on room temperature and frozen samples, and incremental creep test results on consolidated - undrained triaxial samples are presented. This report contains no conclusions or recommendations as there are still two final reports to be issued on this project.

18. Key Words	19. Distribution Statement
Soil, mechanical properties, subgrade, roadbed, rail, train dynamic, loads, static, cyclic, triaxial tests, simulation	A limited number of copies available from the Transportation Development Centre

20. Security Classification (of this report)	21. Security Classification (of this page)	22. No. of Pages	23. Price
Unclassified	Unclassified	160	—

FORMULE DE DOCUMENTATION POUR RAPPORT

1. N° du rapport Transports Canada	2. N° de l'étude — CDT	3. N° de catalogue du destinataire
TP 7215E	4567	

4. Titre et sous-titre	5. Date du rapport
EVALUATION OF MECHANICAL PROPERTIES OF THREE CANADIAN SOILS FOR USE AS RAILROAD SUBGRADE MATERIAL	février 1985
	6. N° du rapport de l'organisme exécutant 184

7. Auteur(s)	8. N° de dossier Transports Canada
Professor R.N. Yong, Dr. P. Boonsinsuk, Mr. F. Caporuscio	D1465-108-6-9

9. Nom et adresse de l'organisme exécutant	10. N° de dossier — ASC
Centre de recherche géotechnique de Université McGill Montréal, Québec	11SD.T8200-1-1572
	11. N° de contrat — ASC ou Transports Canada OSD81-00186

12. Nom et adresse de l'organisme parrain	13. Genre de rapport et période visée
Centre de développement des transports (CDT) Complexe Guy Favreau 200 ouest, boul. Dorchester, Suite 601, Tour Ouest Montréal, Québec H2Z 1X4	1979-1984
	14. Code de l'organisme parrain CDT 4567

15. Remarques additionnelles	16. Agent de projet — CDT
Le CN et le CDT se sont partagés également les coûts de ce projet.	D.W. Dibble

17. Résumé

Ce programme, visant à établir une méthode de calcul de l'épaisseur à donner aux couches granulaires devant composer l'infrastructure d'une voie ferrée, a été lancé en 1979 par le Centre de recherche du CNR. On a fait appel aux services du Centre de recherche géotechnique de l'Université McGill pour la mise à l'essai en laboratoire de trois types de sols, soit deux argiles de la région du lac Edmonton et un limon sableux contenant une faible proportion d'argile provenant de Joliette au Québec.

Le présent rapport rend compte des essais effectués en laboratoire dans le but de comprendre le comportement du sol sous l'effet des charges de circulation. Il donne la composition minéralogique et les caractéristiques des sols essayés ainsi que les résultats des essais triaxiaux statiques consolidés-non drainés, des essais triaxiaux à charge cyclique exécutés sur des éprouvettes à la température ambiante et gelées et des essais de fluage sous charge progressive exécutés sur les éprouvettes ayant servi aux essais triaxiaux consolidés-non drainés. Il ne présente toutefois aucune conclusion ni aucune recommendation puisque deux autres rapports finals doivent encore être préparés.

18. Mots-clés	19. Diffusion
Sols, propriétés mécaniques, plate-forme, infrastructure, voie ferrée, charges de circulation, essais triaxiaux, simulation des conditions d'exploitation	Le Centre de développement des transports dispose d'un nombre limité d'exemplaires.

20. Classification de sécurité (de ce rapport)	21. Classification de sécurité (de cette page)	22. Nombre de pages	23. Prix
non classifié	non classifiée	160	—

EXECUTIVE SUMMARY

This report was prepared by the Geotechnical Research Centre of McGill University under contract to CN Technical Research Centre, and probably documents one of the most extensive laboratory studies ever undertaken in an effort to understand the dynamic behaviour of railroad track subgrade materials under simulated North American train loadings. Uneven roadbed settlements lead to gradual deterioration of track running surface, and track surfacing maintenance costs form a permanent portion of all railroad Engineering budgets. While 100 ton cars have been running in Canada for over 20 years, the full brunt of their loadings and ever increasing numbers is now being felt by all components of track and roadbed.

In 1979, CN Technical Research Centre initiated a study aimed at determining the granular depth design requirements for railroad tracks with different subgrade soil conditions, traffic densities, and axle loading spectra. This report presents laboratory soil test data only, contains no conclusions or recommendations, and is only one of a series of reports detailing both laboratory and field test measurements. Reports published to date are listed in the bibliography at the end of this report. There are two more reports still to be issued on this project, hopefully within the next five months, and it is not known at this time whether or not a successful conclusion will be reached. All of the voluminous information meticulously gathered, both in the laboratory and out on track, has or will be published in order that it may be utilized in any way seen fit by various research agencies and North American railroads.

Executive summary only written by

J.F. Scott
Senior Research Engineer
CN Technical Research Centre

SOMMAIRE

Ce rapport, rédigé par le Centre de recherche géotechnique de l'Université McGill pour le compte du CN, s'inscrit dans l'un des plus importants programmes jamais entrepris pour étudier, dans des conditions simulant les condition réelles d'exploitation en Amérique du Nord, le comportement des matériaux d'assise d'une voie ferrée sous l'effet des charges de circulation. Le tassement différentiel de l'assise d'une voie ferrée entraîne inéluctablement une dégradation progressive de celle-ci en surface également. C'est pourquoi des crédits sont prévus pour l'entretien de cette dernière dans tout budget de construction ferroviaire. Bien que les voies ferrées du Canada soient soumises à la circulation de wagons de 100 tonnes depuis plus de 20 ans, c'est maintenant que les effets des charges, de plus en plus importantes et de plus en plus fréquentes, se font le plus sentir sur tous les éléments de la voie ferrée, comme sur son assise.

En 1979, le Centre de recherche du CN a entrepris un étude visant à établir une méthode de calcul de l'épaisseur à donner aux couches granulaires devant composer l'infrastructure d'une voie ferrée, en fonction des caractéristiques du sol d'assise, de l'intensité du trafic et des spectres de charges des essieux. Le présent rapport ne donne que les résultats des essais effectués en laboratoire sur des éprouvettes de sol; il ne contient ni conclusions, ni recommendations. Il fait partie d'une série de rapports contenant des mesures prises en laboratoire et in situ; la liste des rapports publiés se trouve à la fin du présent rapport. Deux autres rapports sont attendus dans les cinq mois prochains. Il n'est pas possible, à l'heure actuelle, de dire si les recherches porteront fruit. Quoi qu'il en soit, toutes les données si soigneusement recueillies en laboratoire comme in situ seront publiées, si ce n'est déjà fait, afin qu'elles puissent servir aux divers organismes de recherche ainsi qu'aux compagnies ferroviaires nord-américaines.

Sommaire rédigé par

J.F. Scott
Chercheur en chef
Centre de recherche du
CNR

TABLE OF CONTENTS

	Page No.
1. INTRODUCTION	1
2. SCOPE OF TESTING PROGRAM	1
3. TEST PROCEDURES	2
4. CYCLIC TRIAXIAL TESTS (CONSOLIDATED-UNDRAINED) EQUIPMENT AND TEST PROCEDURE	5
5. CYCLIC TRIAXIAL TESTS ON FROZEN SOIL SAMPLES	6
6. INCREMENTAL CREEP TESTS (CONSOLIDATED-UNDRAINED TRIAXIAL) ...	7
7. PRESENTATION OF RESULTS	7
TABULATED AND PLOTTED LABORATORY TEST DATA	18-151
BIBLIOGRAPHY (PREVIOUSLY UBLISHED REPORTS IN SERIES)	152

EVALUATION OF MECHANICAL PROPERTIES OF THREE CANADIAN SOILS FOR USE AS RAILROAD SUBGRADE MATERIAL

1. INTRODUCTION

This report is a supplement to other reports[1] on the design of ballast under Canadian soil conditions (GRC, 1984). The main theme of this report concentrates on the experimental work performed for the selected Canadian soils. All laboratory results obtained in relation to understanding the soil behaviour under train loadings are accumulated herein. The main approach taken in the investigation is the cyclic triaxial test which simulates the soil responses under train wheel loadings. The results are used directly in the analysis of the field measurement data and the design of ballast thickness.

2. SCOPE OF TESTING PROGRAM

A series of static, dynamic and creep triaxial tests was conducted on three Canadian soils. Two different composite clay samples from the Edmonton area were prepared by mixing samples from test pits 2A, 14 and 16 (designated as soil "A") and samples from test pits 9 and 25 (designated as soil "B"). The third soil was retrieved from Joliette, Quebec. The three soils were subjected to standard soil classification tests as listed in Table 1. The mineralogy of samples "A" and "B" was also determined for identification of potential swelling clay.

Consolidated-undrained triaxial tests for determination of soil strength parameters were conducted on all three soils. Cyclic triaxial tests were first performed using sine pulse wave loading

(1) See Bibliography on Page No. 152.

pattern while the truck pulse was later adopted in determining the responses of the soil specimens under dynamic load conditions. The truck pulse shape more closely simulates the double axles at either end of the train cars.

3. TEST PROCEDURES

3.1 Basic Soil Properties

The basic mechanical properties of the three soils were determined according to the following ASTM procedures:

Particle size analysis of soils	D422
Test for liquid limit of soils	D423
Test for plastic limit and plasticity index of soils	D424
Specific gravity of soils	D854
Laboratory determination of moisture content of soils	D2216
Moisture-Density relations of soils using 2.49 kg rammer and 305 mm drop	D698

3.1.1 Mineralogy

An amount of 0.100 g of oven-dried and ground soil sample was weighed and placed into a 10 ml volumetric flask, then dispersed in distilled water until it reached the 10 ml mark.

Two mls of the solution were then pipetted onto each of two slides and allowed to dry. One slide was then run "as is" and the other was heated to 550^{o}C in a thermolyne furnace for 1 hour. The "as is" slide was subsequently run in the diffractometer after treatment with ethylene glycol.

All slides were run on a Siemen's X-Ray Diffractometer using CuKα radiation and the purpose of each method was as listed below.

Method	Purpose
"as is"	general mineral characteristics
550°C	1) differentiate between kaolinite and chlorite 2) detect the presence of mica
ethylene glycol	detection of swelling clay minerals

3.2 Preparation of Triaxial Test Specimens

The triaxial test specimens were compacted in a mould formed by connecting two standard compaction moulds (102 mm diameter by 116.4 mm high) together to prepare a soil cylinder 102 mm in diameter by 204 mm high. The soil was first passed through a No. 4 (4.75 mm) sieve and then compacted according to the standard Proctor criteria. Since the specimen was made up of a total of six layers, the top of each layer was scarified to a depth of approximately 3 mm prior to compaction of the next layer in order to reduce the discontinuity between the layers.

Each compacted specimen was then fitted with side, top and bottom filter drains to reduce pore pressure equalization time, enclosed in a latex membrance (0.38 mm thick) and installed in a triaxial cell with provision for both top and bottom drainage. The triaxial cell was filled with water only up to the top of the specimen, to provide a compressible air layer, thereby allowing for dynamic motion of the loading piston without significantly changing the cell pressure. An

external volume change burette was connected to the drainage lines and the water level in the burette was maintained equal to the water level in the triaxial cell. Therefore, the hydrostatic pressure distribution due to water in the cell was balanced by the hydrostatic pore pressure distribution within the soil. This factor was important because of the very small confining pressures applied to the soil specimens.

Each sample was first saturated by back pressure until the pore water pressure parameter "B" value of 0.95 to 1.00 was obtained. Subsequently, the sample was consolidated to the designated confining pressure (σ_3), allowing it to consolidate beyond the t_{100} value (100% consolidation).

3.3 Static Consolidated-Undrained Triaxial Tests

Static triaxial compression tests were performed on all three soil types prepared as per section 3.2. In all the tests, back pressure was applied in 34.5 kPa increments up to 207-275.8 kPa to bring the "B" pore pressure parameter to 0.95-1, which indicates a degree of saturation near 100%. Isotropic consoldiation pressure was then applied in order to consolidate the sample under pressures of 13.8, 27.6 and 41.4 kPa. The specimens were then loaded in a constant deformation testing machine using a strain rate value of 0.0508 mm/min (1.5% per hour). The consolidation data indicated that this rate was sufficiently low to allow 95% equalization of pore pressure.

The load, pore water pressure and displacement were all measured and recorded continuously by the data acquisition system. After failure the samples dimensions were measured and final water content taken.

4. CYCLIC TRIAXIAL TESTS (CONSOLIDATED-UNDRAINED) EQUIPMENT AND TEST PROCEDURE

The dynamic triaxial tests were performed using a closed loop electro-hydraulic servo system. A function generator supplied a reference voltage to a load generator which in turn activated the servo valve controller. The servo valve admits or releases oil from a hydraulic loading ram which is connected through a load cell to the triaxial cell piston. The controller compares the reference voltage from the function generator with the load cell output and adjusts the position of the servo valve such that the load cell output follows the form of the reference voltage. Both a sine function and/or a truck pulse were used where the dynamic deviator stress (σ_1-σ_3) applied to the sample varied between zero and the present peak deviator stress. The rate of loading was kept constant at 1 cycle per second. Figure 1 illustrates the schematic diagram of the dynamic triaxial system, while Figs. 2-2A show the two different wave forms used in testing.

Upon application of the dynamic deviator stress (σ_1-σ_3) the strains, both cumulative and elastic, were measured with displacement transducers linked to strip chart recorders. The pore water pressure measured at the base of the sample was also recorded with a chart recorder. The load cell could either be monitored on an oscilloscope and/or recorded by a chart recorder.

5. CYCLIC TRIAXIAL TESTS ON FROZEN SOIL SAMPLES

The confining pressure that was applied to soil "A" was 13.8 kPa and unlike the other test series, this was not varied. During the previous test series, the triaxial tests were carried out at three different confining pressures of 13.8, 27.6 and 41.4 kPa. The highest vertical stress with the lowest confining pressure in the ballast structure lies just below the granular layer in the subgrade. The confining pressure in the subgrade for a ballast depth of 635 mm is found to be around 10.3 kPa. In order to facilitate comparison with the previous tests, a confining pressure of 13.8 kPa was chosen for all the triaxial tests.

Other changes in the testing procedure included varying the testing condition in order to model the effects of seasonal changes. Therefore, in this series of experiments the samples were tested frozen and thawed. In the latter state the samples were tested first in the unconsolidated state (UU-test) and subsequently in the consolidated state (CU test).

The test samples were frozen prior to testing and cyclic load application was begun in the frozen state. At approximately a thousand cycles the cyclic loading was stopped and the samples were allowed to thaw with no drainage. After complete thawing the load application was resumed for a limited number of cycles ($\approx$ 200). Drainage was allowed with no cyclic load and therefore the sample was allowed to consolidate under a confining pressure (σ_3) of 13.8 kPa. Upon completion of the consolidation stage, the drainage valves were closed and cyclic loading was resumed (CU test) until failure or 10^6 cycles in the case of equilibrium.

6. INCREMENTAL CREEP TESTS (CONSOLIDATED-UNDRAINED TRIAXIAL)

Prediction of the behaviour of the clay under dynamic loading was attempted through creep tests. The relationship between the two methods of loading has been reported by Hyde, et al. (1976). If the formulation of a definite relationship is successful it will be possible to predict the behaviour of the subgrade in the field under repeated train loadings through creep tests, thereby eliminating the need of sophisticated test equipment.

The sample preparation discussed in Section 3 was adopted. The procedure for consolidated undrained triaxial tests (section 3.3) was followed in saturating and consolidating the test specimens.

After consolidation, the axial stress was increased to the required levels by a hanger system, after which the sample was allowed to creep until negligible change or failure occurred. The deviator stress was applied in 20% (of final load) increments.

7. PRESENTATION OF RESULTS

7.1 Basic Mechanical and Mineralogical Properties

The basic mechanical properties of the three soil samples are listed in Table 2 while the grain size and compaction curves are shown in Figs. 3-7. It is evident from the grain size distributions and values of Atterberg limits that soils "A" and "B" are silty clay soils with little sand, while the Joliette soil is a sandy silt with little clay.

Mineralogical identification for soils "A" and "B" of particles less than 2 μm as determined by X-ray diffraction are given below.

It is clear that such factors as permeability and swelling behaviour will be significantly influenced by the presence of montmorillonite.

Mineral	% of Total Minerals	
	Soil "A"	Soil "B"
Montmorillonite	10.1	13.8
Kaolinite	38.5	37.9
Chlorite	0.5	0.5
Illite	50.9	47.8

7.2 Static Triaxial Comoression Tests (Consolidated-Undrained)

The static triaxial compression tests performed in order to establish the soil strength parameters are listed in Tables 2 and 3. The stress path diagrams, deviator stress and pore pressure relationships are illustrated in Figs. 8-16.

It should be noted that specimens for soil "A" and soil "B" failed with well-developed shear planes. The pore pressures reached a peak value initially and then declined which is a characteristic of compacted materials tested at low confining pressure.

The Joliette soil specimens bulged during failure while the pore water pressure peaked and then negative pore water pressure was observed.

The strength parameters obtained from the static tests are:

Soil Type	c' (kPa)	ø' (deg)
Soil "A"	13.8	22.6
Soil "B"	3.5	27.1
Joliette	0	42.0

7.3 Dynamic Triaxial Test Using Sine Pulse Loading

7.3.1 Soil "A" (Undrained Condition)

Table 5 shows the parameters for different dynamic tests conducted on specimens prepared from composite sample "A". Figure 17 shows the relationship between cumulative axial strain and number of load cycles for specimens tested at 13.8 kPa confining pressure under undrained conditions. There are several features to note here:

1. with the exception of test no. 7, the slope of the cumulative strain versus log number of cycles is relatively constant
2. a reduction of the peak deviator stress by 50% (test no. 4 versus test no. 7) produces very small cumulative strain values.
3. British Rail procedures would suggest that a stress level of approximately 41.1 kPa (deviator stress) would lead to an equilibrium situation at about 1,000,000 cycles where no further strain would be anticipated.
4. the threshold stress is between 41.4 kPa and 44.1 kPa

The elastic strain versus number of cycles relationship is shown in Fig. 18 for undrained tests at 13.8 kPa confining pressure. A threshold elastic strain of about 0.5% is obtained where failure is not anticipated if strain levels are maintained below this limit. It should be noted that elastic strain values decrease with increasing number of load cycles. This behaviour may be related to gradual reduction in pore water pressure following initial increase shown in Fig. 19. The reduction in pore water pressure is consistent with the dilatant nature of the material as shown in the static tests. Values

of dynamic moduli are given in Fig. 20. These values are the ratio of dynamic stress (kPa) to dynamic strain (mm per mm).

Figures 21 through 28 show the test results performed at two different confining pressures of 27.6 and 41.4 kPa under undrained conditions. The reason for performing these tests was to show the effect of confining pressure, which is a function of depth in the subgrade, on the dynamic behaviour of soil.

Increasing the confining pressure has a considerable effect on the threshold stress level. While in the case for the 13.8 kPa confining pressure the threshold stress level was in the range of 41.4 kPa, the threshold stress increased to about 57 kPa and 69 kPa for confinng pressures of 27.6 and 41.4 kPa, respectively.

Under the higher confining pressures, the threshold elastic strain was reduced from 0.5% (13.8 kPa confining pressure) to about 0.4%.

7.3.2 Soil "A" (Drained Condition)

Consolidated-drained dynamic triaxial tests were conducted on soil "A" at a confining pressure level of 13.8 kPa. The information obtained assists in developing a better understanding of the subgrade dynamic response behaviour under different drainage conditions.

Figure 29 shows the relationship between cumulative strain and number of cycles for the drained tests. It is noted that under such drained test conditions, increase in the threshold stress is obtained. A cyclic deviator stress of about 49.6 kPa, which is higher than that observed in the undrained condition, can be applied without excessive strain, even after a high number of cyclic load applications.

Figures 30 and 31 illustrate the variation of the elastic strain and the dynamic modulus with the number of cycles, respectively. The threshold elastic strain has dropped to about 0.3%. The increase of volume at the onset of failure is shown in Fig. 32 and is consistent with the decrease in measured pore water pressure values obtained in the undrained tests.

The actual field conditions are anticipated to be neither completely undrained nor fully drained. Therefore, a periodically drained test (No. 16) was conducted where drainage was allowed for a time period of 30 minutes (i.e. 1800 load cycles) during which no cyclic load was applied, simulating the rest period between passing trains in the actual field case. Figure 33 shows that this test exhibits rather unusual pore water pressure behaviour. The pore water pressure values shown are the maximum pressure developed during the undrained load cycling phase.

7.3.3 Soil "B" (Undrained Condition)

Table 6 shows the test parameters investigated for specimens prepared from soil "B". Figure 34 shows the development of cumulative strain with number of load cycles. The threshold stress for soil "B" at 13.8 kPa confining stress is approximately 38.6 kPa. Figure 35 suggests that the elastic strain level is about 0.25%. The pore water pressure trends established for soil "A" seem to be reflected in soil "B" as shown in Fig. 36, including the behaviour of dynamic modulus (Fig. 37).

7.4 Dynamic Triaxial Test Results with Truck Pulse Loading Pattern

7.4.1 Soil "A" (Undrained Condition)

Table 7 shows the test results for different dynamic tests using the truck pulse load sequence on specimens prepared from composite sample "A". Figures 38 to 49 show the various relationships with the number of cyclic load cycles. The features to note are as follows

1. The threshold stress for confining pressures of 13.8, 27.6 and 41.4 kPa are 34.5, 45.5 and 51.7 kPa respectively.
2. The threshold stresses obtained from the truck pulse are lower than those from the sine pulse

Confining Pressure (kPa)	Threshold Stress (kPa)	
	Sine Pulse	Truck Pulse
13.8	41.4	34.5
27.6	57.0	45.5
41.4	69.0	51.7

3. The threshold elastic strains obtained from Figs. 39, 43 and 47 are listed below

Confining Pressure (kPa)	Threshold Elastic Strain (%)
13.8	0.30
27.6	0.20
41.4	0.25

The threshold elastic strain decreases for confining pressures above 13.8 kPa. This behaviour is similar to the data obtained from the sine pulse.

4. The pore water pressure behaviour is similar to the sine pulse test results. The reduction in pore water pressure is consistent with the dilatant nature of the material.

7.4.2 Soil "B" (Undrained Condition)

Table 8 summarizes the test results for soil "B" cycled with the truck pulse while Figs. 50-61 show the various relationships with the number of truck pulse cycles.

The following points should be noted:

1. When soil "B" was loaded repetitively with the sine wave under a confining pressure of 13.8 kPa the threshold stress was 38.6 kPa. The threshold stress for truck pulse under the same confining pressure of 13.8 kPa is 37.2 kPa. The decrease in threshold stress is therefore not significant.
2. The threshold elastic strain ranges approximately between 0.2% and 0.3%. This range is consistent with the sine pulse test data.
3. The pore water pressure trends established for the sine pulse test data seem to be reflected in the truck pulse results, i.e. gradual reduction in pore water pressure following initial increase which is consistent with the dilatant nature of the material as shown by the static tests.
4. Except for the confining pressure of 27.6 kPa, the threshold stresses obtained are as follows:

Confining Pressure (kPa)	Threshold Stress (kPa)
13.8	37.2
27.6	not available
41.4	48.3

The increase in threshold stress with increase in confining pressure is in agreement with the values obtained for soil "A" using both sine and truck pulse.

7.4.3 Joliette Soil (Undrained Condition)

The dynamic properties of the Joliette soil are listed in Table 9 and illustrated in Figs. 62-73. The major features to note are

1. The samples neither failed in shear nor liquified but instead bulged excessively
2. The threshold stress can hardly be identified, primarily due to its silty soil characteristics.
3. The static tests normally failed at approximately 3% cumulative strain. Assuming failure occurring at about 3% strain in the dynamic triaxial tests, the following approximations in the values of threshold stress (for comparison with other soils) can be made:

Confining Pressure (kPa)	Approximate Threshold Stress (kPa)
13.8	22.1
27.6	34.5
41.4	44.8

If the slopes of the cumulative strain versus number of cycles are considered it is clear that the slope of the curve changes drastically as the cyclic deviator stress is increased. If the threshold stresses are obtained in this manner the values are as follows

Confining Pressure (kPa)	Threshold Stress (kPa)
13.8	22.1
27.6	27.6
41.4	44.8

4. The elastic strain generally decreases with the number of load cycles. The average elastic strain is approximately 0.3% except for the confining pressure of 27.6 kPa.
5. The pore water pressure, after the initial increase, tends to remain constant compared to the static tests, the pore water pressure, after the small initial increase, showed negative pressures.

7.4.4 Soil "A" (Frozen)

The standard cyclic triaxial test procedure was modified in order to study the effect of seasonal changes through the use of frozen and thawed samples. The results of the tests are shown in Fig. 74-78. The test samples were frozen prior to testing and load application was commenced in the frozen state. During the frozen state the deformation in the soil is minimal and the whole sample mass acts rigidly. Significant deformation develops when the clay is thawing, thawed and unconsolidated. The thawed, consolidated portion of the testing exhibits strains which are substantial but not the same magnitude as those

occurring in the previous stage.

The threshold stress for the frozen-thawed and consolidated soil "A" is between 30.3 and 38.6 kPa.

In the field the thawed-unconsolidated period occurs during the spring. Its duration is a function of the particular climatic conditions and subgrade soil properties considered. In relating the characteristics of the spring period to the test conditions the following sequence applied. The advent of winter causes a certain depth of the soil to freeze. With the spring thaw, this frozen layer begins to melt from above, due to warmer air temperatures, and from below, due to the geothermal influence. As a result the soil above the frost line has no drainage path and as such, its properties approach those of the test conditions. In addition, it should be noted that due to the presence of montmorillonite, the soil under test has low permeability which would cause the period of maximum instability to have a longer duration than simply the time for the frozen layer to completely melt. Once the frozen layer has melted completely, the drainage path is free and consolidation occurs. This part of the annual cycle is represented by the consolidated undrained dynamic triaxial test.

7.5 Incremental Creep Tests

The test parameters of the incremental creep tests for all three soils are listed in Tables 10-13, while Figs. 80-119 show the pore water pressure and cumulative strain-time relationships. The table which follows shows the threshold stress obtained from the incremental creep tests:

Soil Type	Consolidation Pressure (kPa)		
	13.8	27.6	>41.4
"A"	27.6	46.7	49.4
"B"	38.4	41.4	49.4
Joliette	N.A.	N.A.	N.A.

When the above data is compared with the threshold strengths obtained from the dynamic tests the following points are noted:

1. For soil "A" the sine pulse threshold strength are consistently greater. The truck pulse threshold strengths are relatively the same as the creep test values.
2. For soil "B" the truck pulse threshold strengths are consistently lower than the creep test values.

The Joliette soil exhibits insignificant cumulative creep strain until 60% creep load. Beyond the 60% load, the amount of creep strain measured is not sufficient to cause failure or excessive deformations.

TABLE 1 Test Format

TEST TYPE	SOIL SAMPLE		
	"A"	"B"	Joliette
Atterberg Limit	X	X	X
Natural Moisture Content	X	X	N.A.*
Specific Gravity	X	X	X
Grain Size Distribution	X	X	X
Moisture Density Relationship	X	X	X
Mineralogy	X	X	N.R.**
Consolidated Undrained Triaxial Test (Static)	X	X	X
Cyclic Triaxial			
Sine Pulse	X	X[1]	N.A.
Truck Pulse	X	X	X
Incremental Creep Test	X	X	X

[1] only for confining stress (σ_3) = 13.8 kPa

* N.A. = not available

** N.R. = not required

TABLE 2 Basic Mechanical Properties

Soil	Natural Moisture Content (%)	Specific Gravity	Standard Proctor		Atterberg Limits			Grain Size Distribution		
			Optimum Moisture (%)	Maximum Dry Density (Mg/m^3)	L.L. (%)	P.L. (%)	P.I.	Sand (%)	Silt (%)	Clay (%)
A	36.0	2.77	24.0	1.574	70.0	28.5	41.5	3.0	44.0	53.0
B	37.0	2.78	25.8	1.535	92.0	35.0	57.0	8.0	25.0	67.0
Joliette	N.A.*	2.61	15.4	1.717	22.5	N.P.**	N.P.	35.0	50.0	15.0

L.L. = liquid limit; P.L. = plastic limit; P.I. = plasticity index

* N.A. = not available

** N.P. = non-plastic

TABLE 3 Consolidated Undrained Triaxial Test Parameters for Specimens Prepared from Composite Sites "A" and "B"

Test No.	Soil Type	Test Condition	Consolidation Pressure (kPa)	Back Pressure (kPa)	Peak Deviator Stress (kPa)	Strain at Peak Deviator Stress (%)	Pore Water Pressure at Peak Deviator Stress (kPa)	Initial Water Content (%)	Final Water Content (%)	Dry Density (Mg/m^3)	Shear Rate (mm/min)
1	A	undrained	13.8	207.0	53.8	25.0	10.2	28.0	32.3	1.50	.051
9	A	undrained	27.6	241.0	71.7	24.0	15.1	28.5	33.2	1.51	.051
32	A	undrained	41.4	275.0	82.7	9.0	18.0	27.5	29.9	1.54	.051
11	B	undrained	13.8	207.0	21.0	4.0	7.7	28.5	42.97	1.45	.051
13	B	undrained	27.6	207.0	35.9	2.0	13.5	28.5	38.07	1.45	.051
15	B	undrained	13.8	207.0	24.8	2.4	8.3	28.5	40.5	1.45	.051
15B	B	undrained	41.4	275.8	71.4	3.0	19.4	28.0	35.0	1.44	.051

TABLE 4 Consolidated Undrained Triaxial Test Parameters for Joliette Soil (Multi-level)

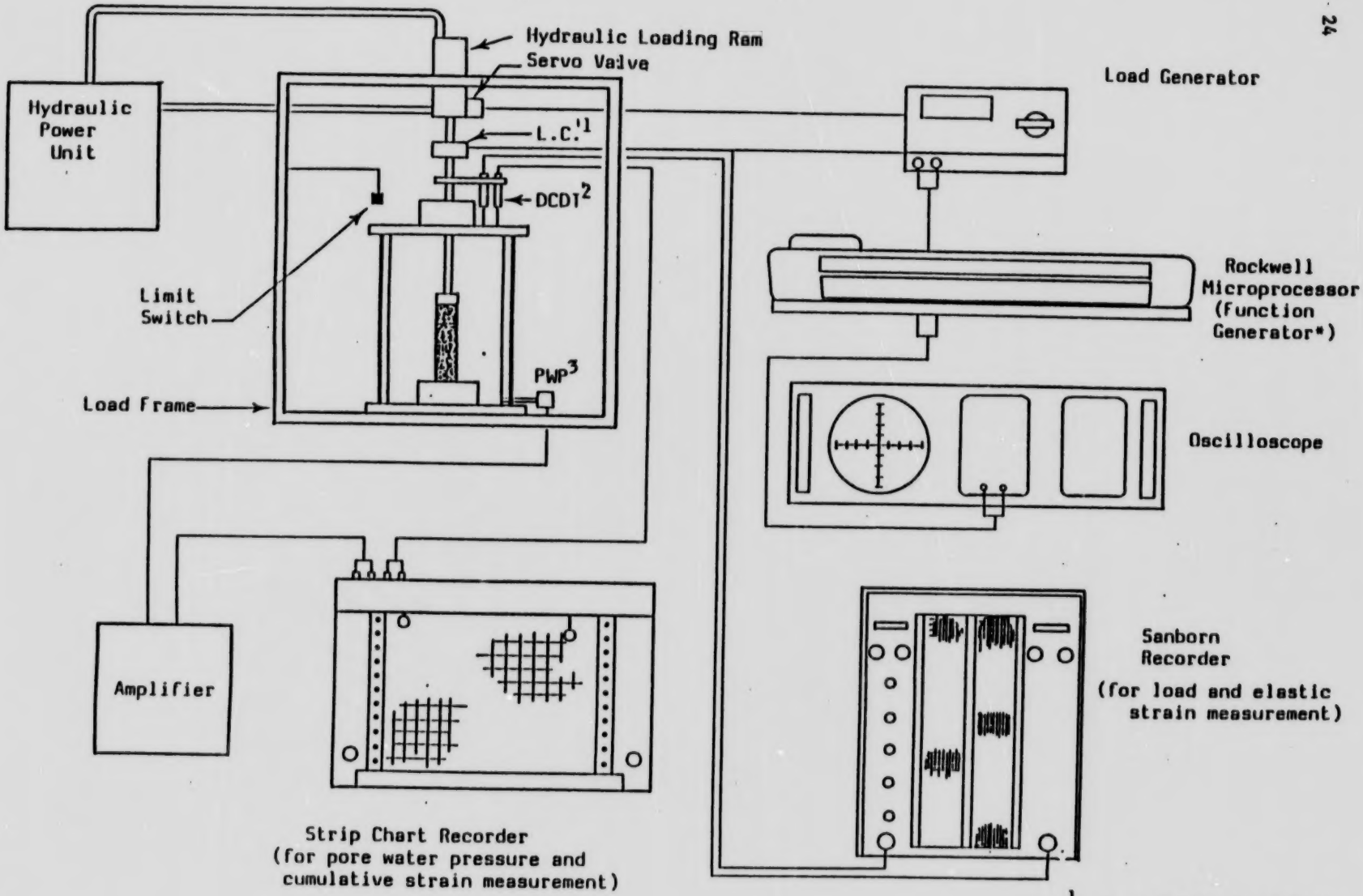
Test Condition	Consolidation Pressure (kPa)	Back Pressure (kPa)	Peak Deviator Stress (kPa)	Strain at Peak Deviator Stress (%)	Pore Water Pressure at Peak Deviator Stress (kPa)	Initial Water Content (%)	Dry Density (Mg/m^3)	Shear Rate (mm/min)
undrained	13.8	275.8	50.0	2.5	1.8	17.0	1.77	0.051
"	27.6	275.8	191.7	3.9	-28.3	17.0	1.77	0.051
"	41.4	275.8	322.2	3.0	-58.5	17.0	1.77	0.051

Test Number	Soil Type	Test Condition	Consolidation Pressure (kPa)	Back Pressure (kPa)	Peak Cyclic Deviator Stress (kPa)	No. of Cycles	Cumulative Strain (%)	Range of Elastic Strain (%)	Range of Dynamic Modulus (MPa)	Range of Pore Pressure (kPa)	Initial Water Content (%)	Final Water Content (%)	Dry Density (Mg/m^3)	Remarks
2	A	undrained	13.8	206.9	27.6	6400	5.65	0.49→0.50	5.5→5.6	.28→1.5	27.7	35.1	1.49	sample failed
3	A	drained	0.0	206.9	11.0	30000	0.04	.065→.068	16.7→16.9	Drained conditions, P.P. measurements non-applicable	28.3	34.0	1.52	sample failed
					22.1	20000	0.07	0.14→0.12	15.5→18.3					
					30.3	22000	0.29	0.2 →0.21	15.5→14.4					
					38.6	13000	3.25	0.36→0.41	9.7→10.3					
					44.1	2400	6.8	0.41→0.52	8.3→10.7					
4	A	undrained	13.8	206.9	20.1	13000	.078	0.09→0.05	24.1→43.4	2.76→2.3	27.5	32.0	1.52	failed
					38.6	27000	.078	0.26→0.3	14.8→12.8	4.0→2.1				
					49.6	18000	2.65	0.54→0.58	8.6→10.3	4.14→4.8				
					55.2	1080	6.3	0.86→1.2	5.5→6.2	0→ -8.96				
5	A	undrained	13.8	206.9	44.1	130000	5.7	0.66→0.50	6.7→8.8	6.48→ -.40	27.4	32.6	1.52	failed
6	A	"	13.8	275.8	38.6	430000	4.5	.3→.345→.275	11.0→15.1	7.72→0	28.0	33.3	1.50	not failed
7	A	"	13.8	275.8	49.6	7800	11.25	.4→.75→.69	6.6→7.17	5.03→-1.93	28.5	32.2	1.50	failed
8	A	"	13.8	241.0	41.4	780000	1.8	0.36→0.17	11.4→24.1	7.17→-7.12	27.52	30.77	1.52	not failed
10	A	"	13.8	241.0	41.4	165000	3.3	0.48→0.27	9.3→15.2	7.45→1.65	27.8	32.48	1.51	not failed
16	A	undrained during cycling -drainage in between cycling for 30 min	13.8	275.8	45.5	21000	14.5	.36→.62→.48	7.6→11.7	8.3→-2.3	26.5	32.2	1.54	failed

TABLE 5 Dynamic Triaxial Test Parameter for Specimens Prepared from Composite Sample "A"

Test Number	Soil Type	Test Condition	Consolidation Pressure (kPa)	Back Pressure (kPa)	Peak Cyclic Deviator Stress (kPa)	No. of Cycles	Cumulative Strain (%)	Range of Elastic Strain (%)	Range of Dynamic Modulus (MPa)	Range of Pore Pressure (kPa)	Initial Water Content (%)	Final Water Content (%)	Dry Density (Mg/m^3)	Remarks
18	A	undrained	27.6	275.8	49.6	1000000	1.64	0.35-0.2	13.8-24.8	9.3-1.24	26.04	29.5	1.55	not failed
19	A	"	27.6		59.3	2400	14.0	0.43-0.3	20.7-13.8	17.9-3.5	26.1	30.0	1.55	failed
21	A	"	27.6		66.2	3400	14.0	0.38-0.25	25.5-17.2	16.6-.35	25.3	28.6	1.56	failed
22	A	"	27.6		55.2	1000000	3.13	0.34-0.28	19.7-16.6	15.5-2.1	26.4	29.4	1.54	not failed
23	A	drained	13.8		44.1	40000	0.31	.098-.091	48.7-44.8		28.0	30.7	1.52	not failed
					56.5	136200	14.0	.32 -.15	36.9-17.6					failed
24	A	"	13.8		49.6	1000000	2.91	0.31-0.17	27.6-15.9		27.9	30.6	1.51	not failed
25	A	undrained	13.8		41.4	220000	0.42	0.11-0.13	36.2-31.5	6.9-4.1	28.3	30.5	1.50	not failed
26	A	"	27.6		57.2	110000	5.0	0.35-0.2	14.5-24.8	22.1-2.8	28.4	29.7	1.53	not failed
27	A	"	41.4		55.2	140000	0.5	0.19-0.14	34.5-29.0	4.8-.35	28.6	31.0	1.48	not failed
28	A	"	41.4		62.1	500000	3.6	0.28-0.16	35.9-24.8	26.6-.1	28.2	29.9	1.51	not failed
29	A	"	41.4		69.0	600000	8.2	0.3 - 0.18	39.3-22.8	14.5-2.1	26.9	29.2	1.56	not failed
30	A	"	41.4		73.8	170000	12.0	0.45-0.23	31.7-16.6		27.3	29.8	1.57	failed
31	A	"	41.4	275.8	75.8	420	14.0	1.02-0.28	19.3-6.9	22.4-0.0	28.0	30.0	1.49	failed

TABLE 5 Cont'd



[1]L.C. = Kulite Load Cell

[2]DCDT = Hewlett Packard displacement transducer

[3]PWP = Statham Pore Water pressure transducer

*Designed/Programmed by McPiepps Reg'd

Fig. 1 Schematic Diagram of Cyclic Triaxial System

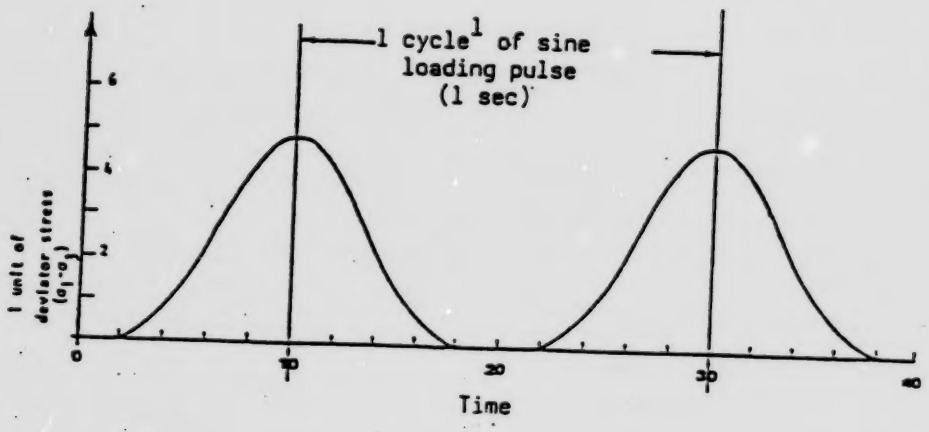


Fig. 2 Sine Pulse Shape

[1]one cycle represents two axles

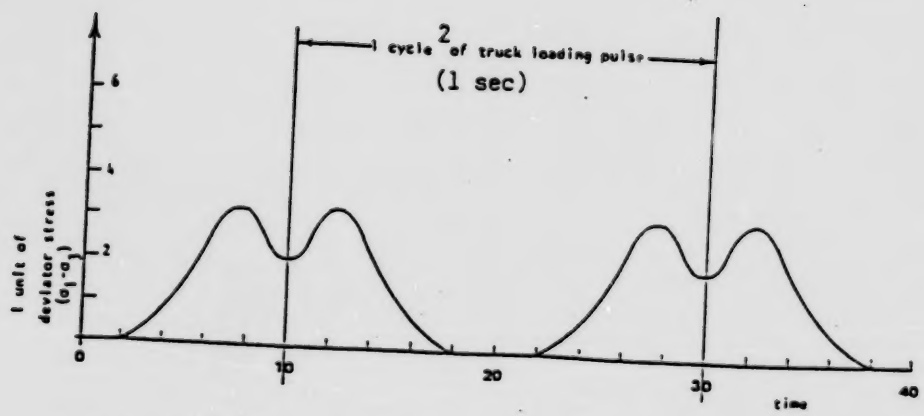


Fig. 2a Truck Pulse Shape to Simulate Actual Field Loading Conditions

[2] one cycle represents two axles (two cycles per standard 4 axle car)

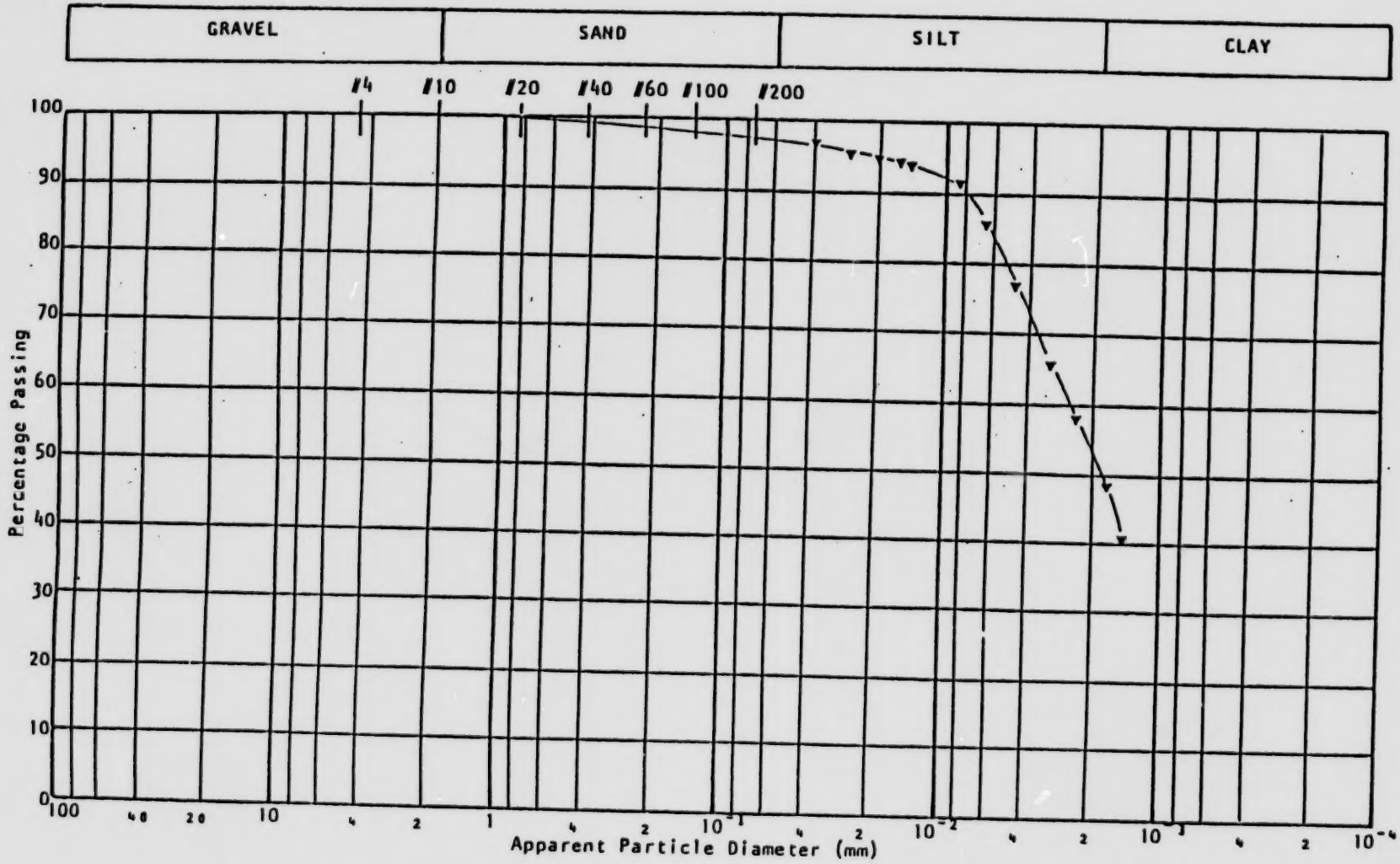


Fig. 3 Particle Size Distribution Curve for Composite Soil "A"

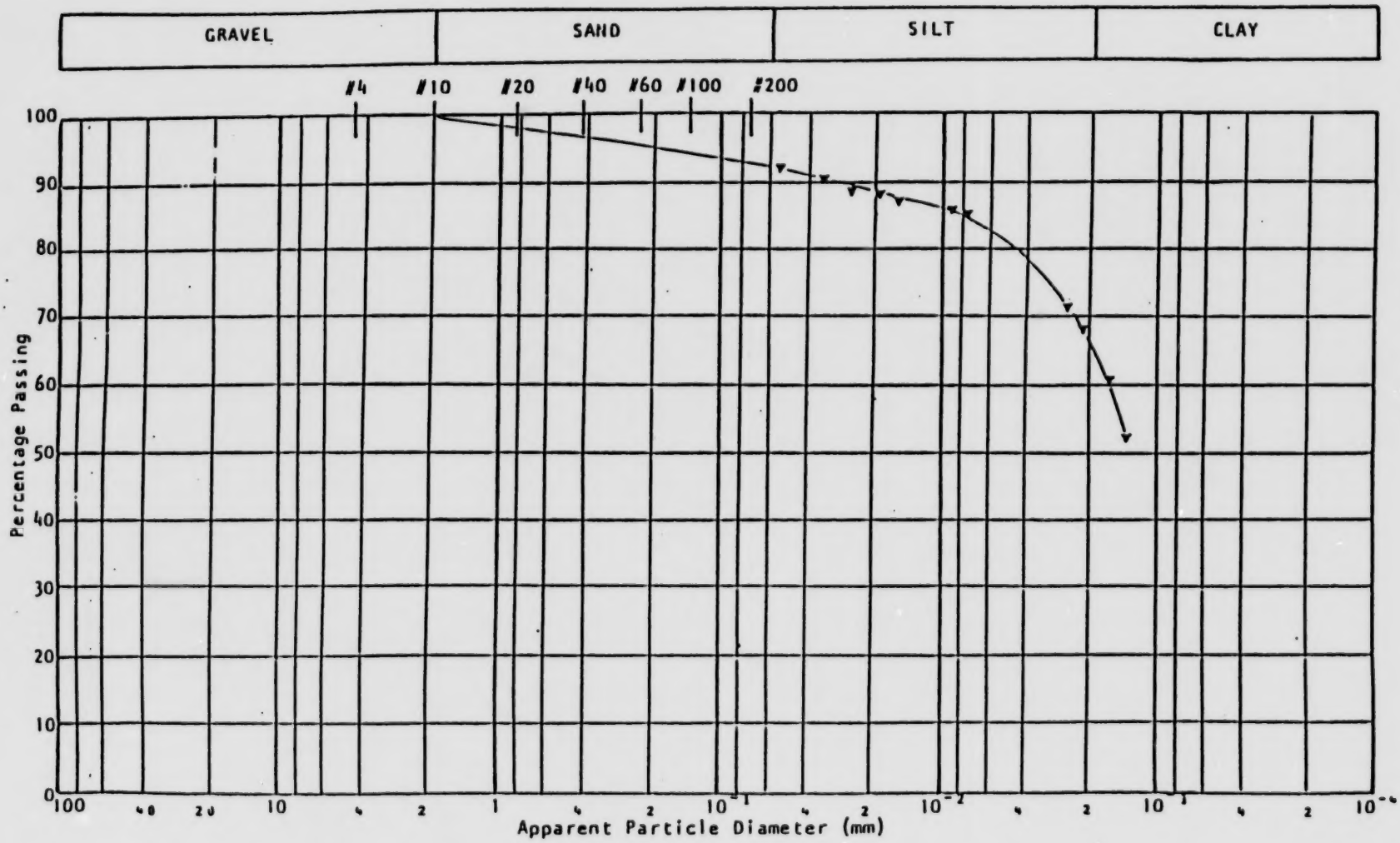


Fig. 4 Particle Size Distribution Curve for Composite Soil "B".

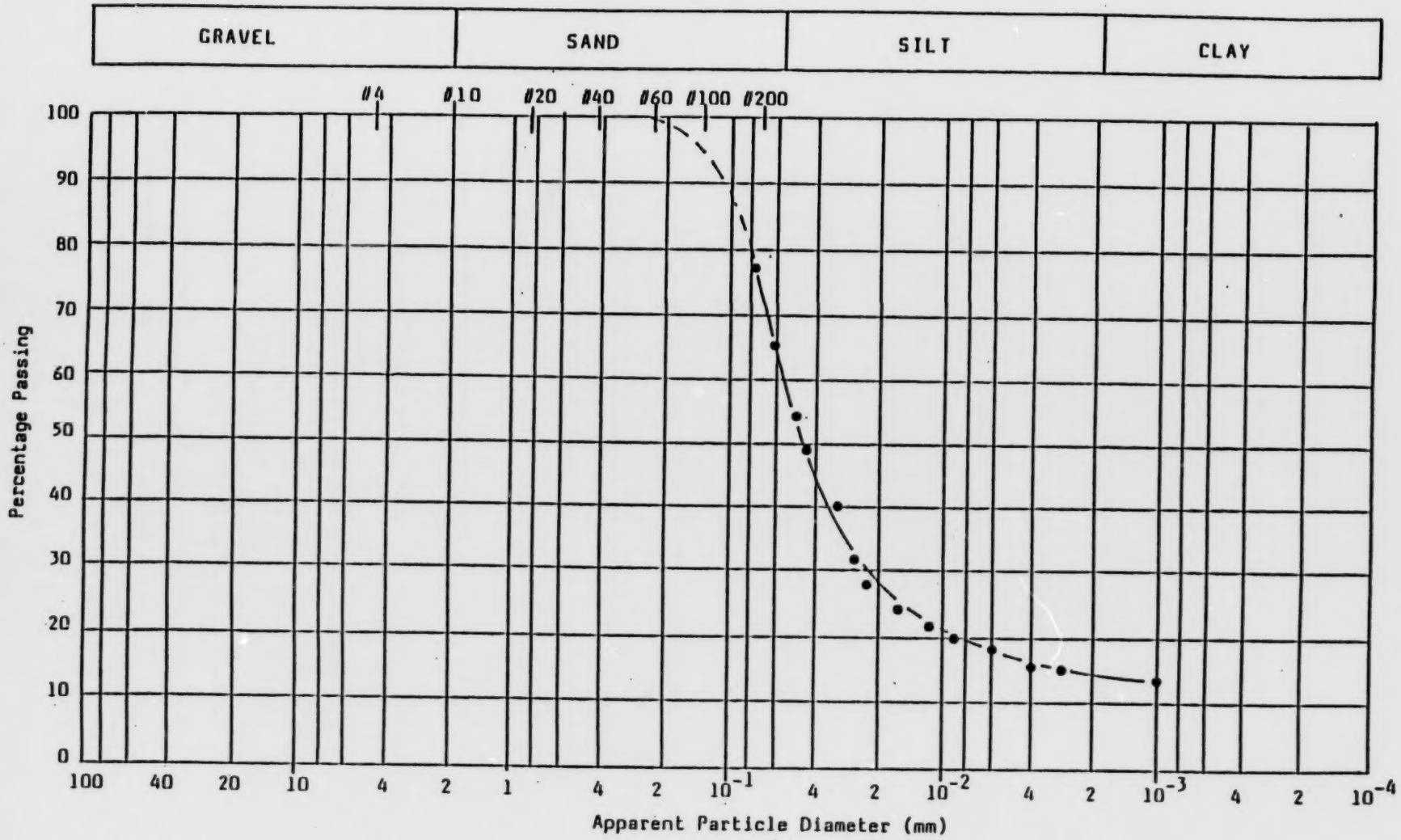


Fig. 5 Particle Size Distribution Curve for Joliette Soil

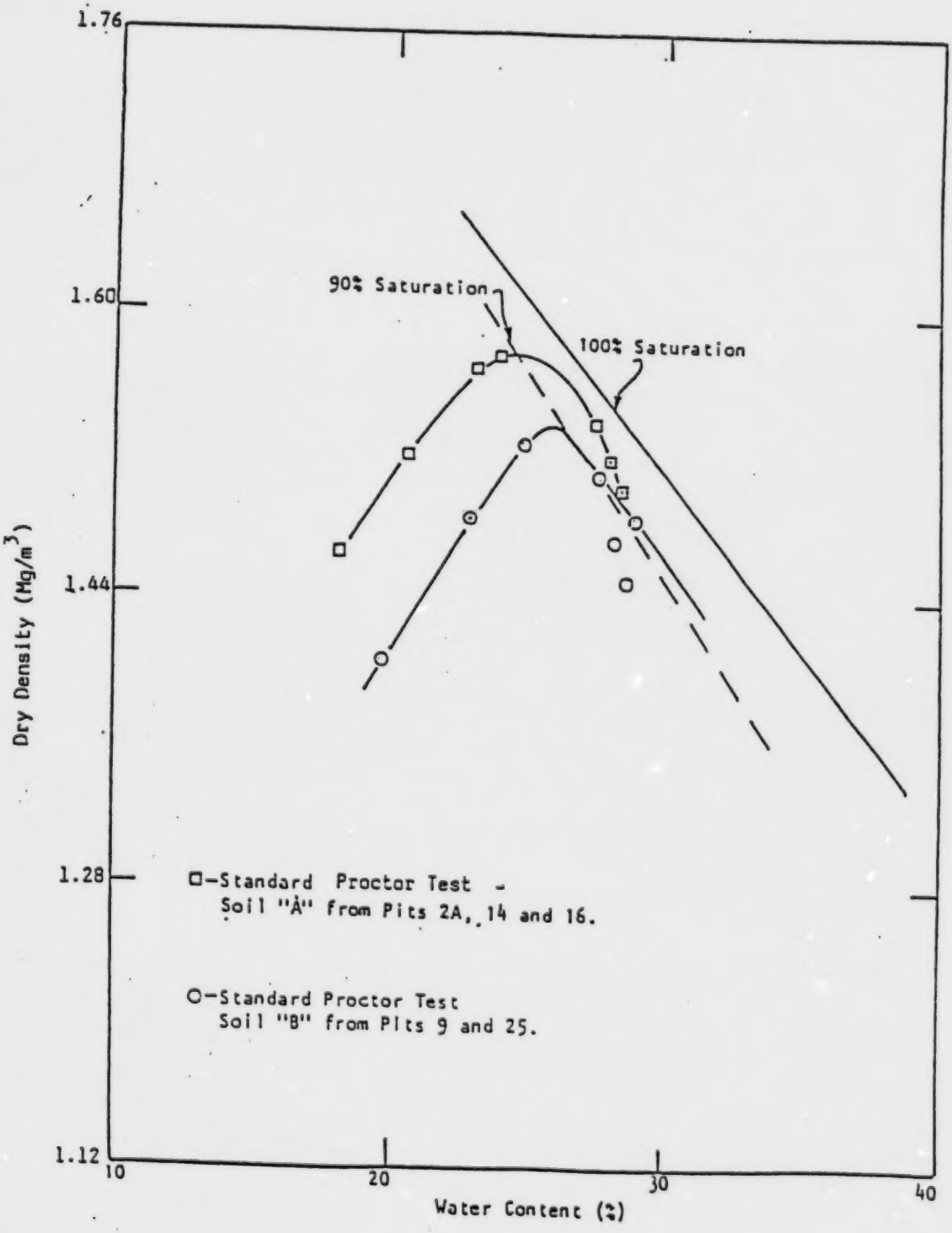
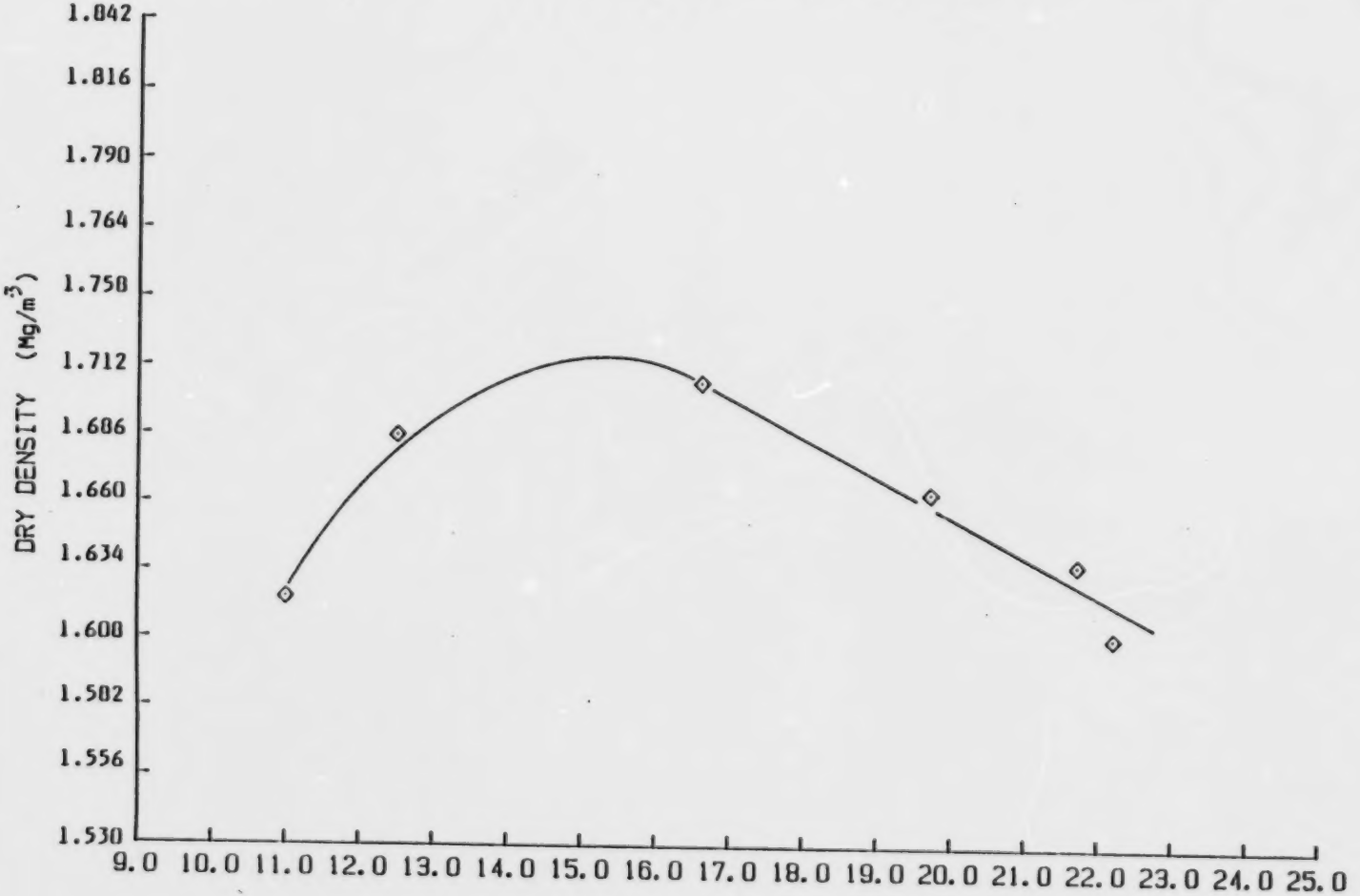


Fig. 6 Standard Proctor Moisture-Density Relationship for Composite Soils "A" and "B"

Fig. 7 Standard Proctor Moisture-Density Relationship for Joliette Soil



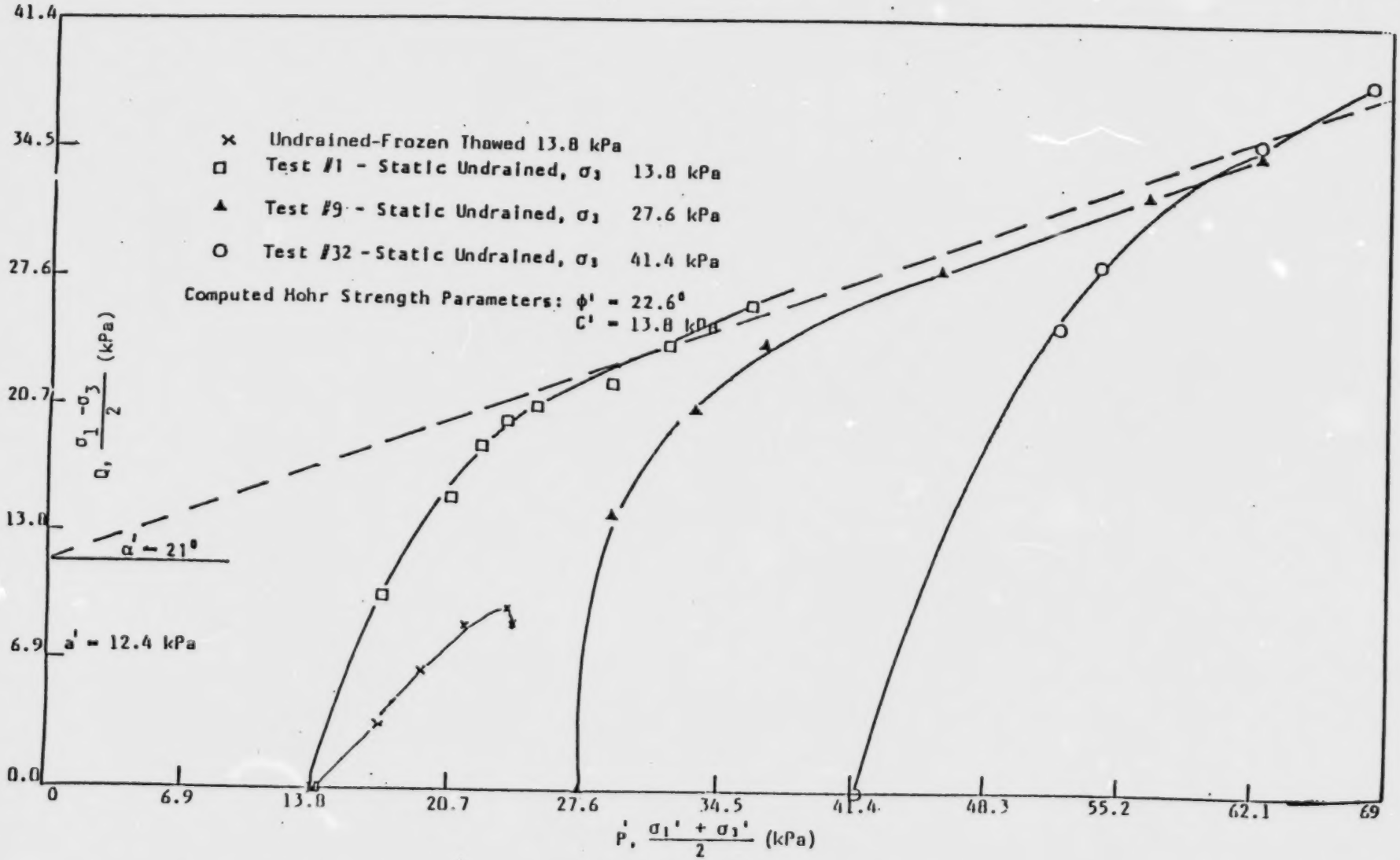


Fig. 8 Stress Path Diagram for Static Tests Performed on Composite Soil "A"

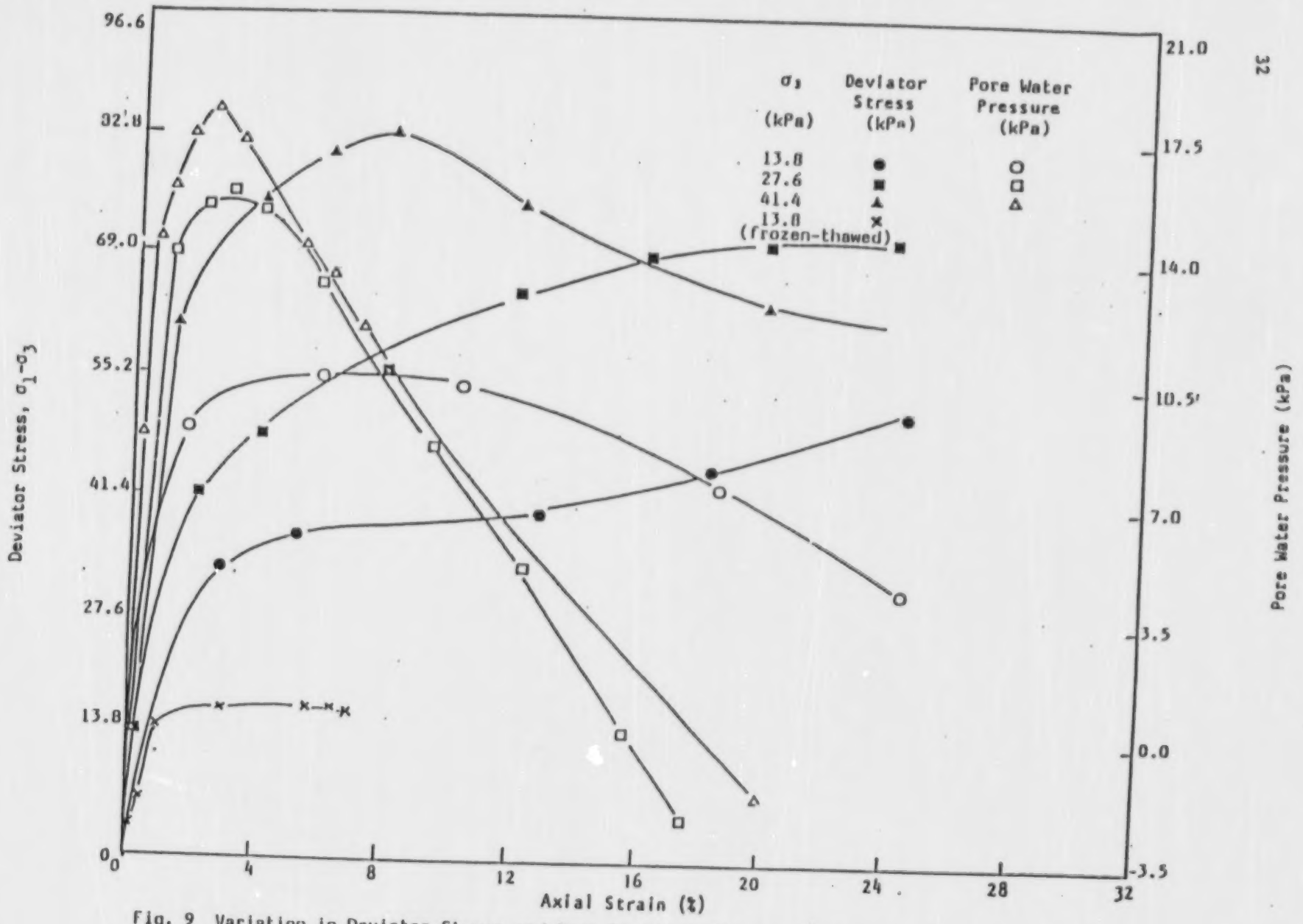
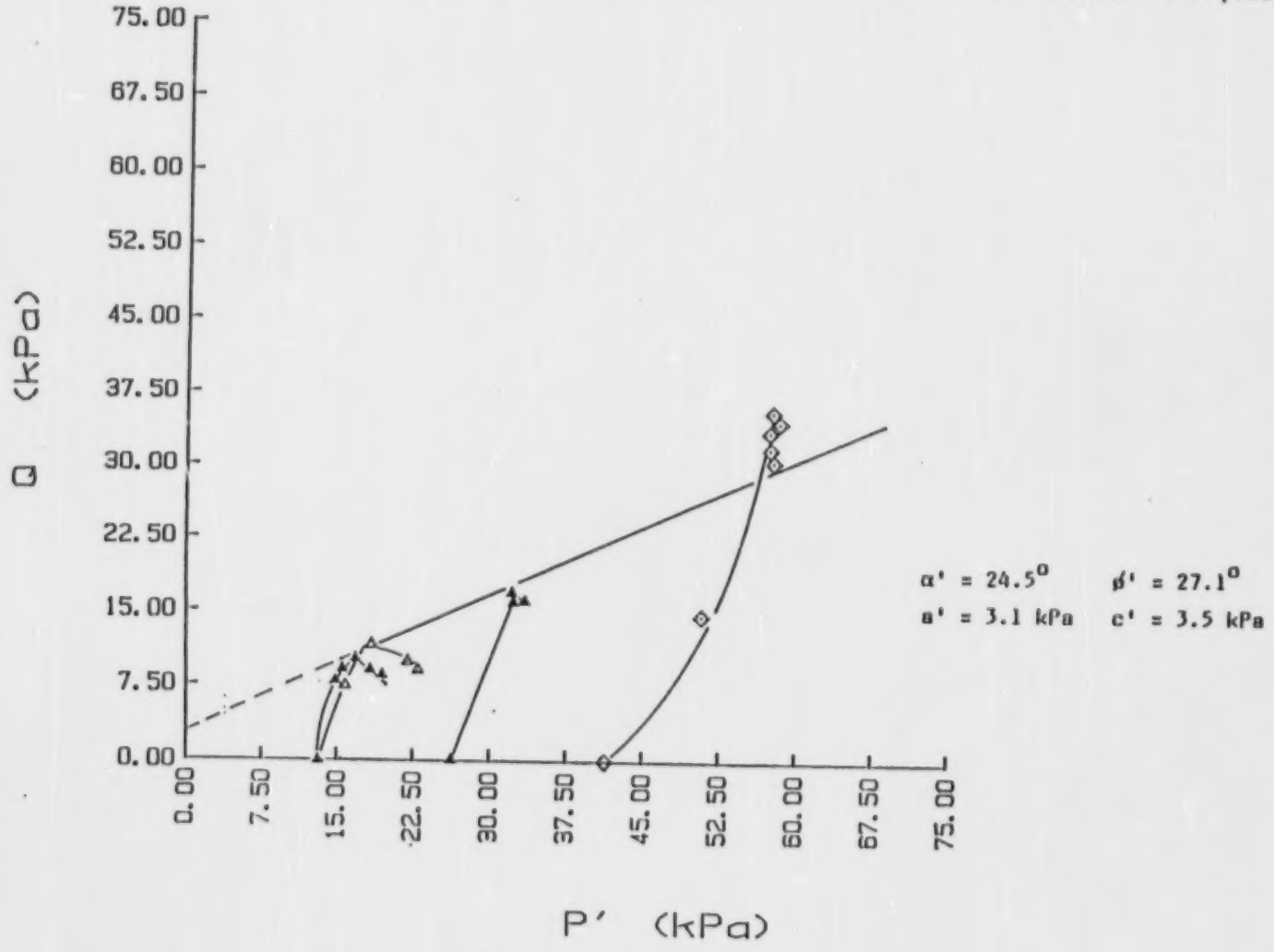


Fig. 9 Variation in Deviator Stress and Pore Pressure with Axial Strain for Composite Soil "A"

Fig. 10 Stress Path Diagram for Consolidated Undrained Triaxial Tests Performed on Composite Soil "B"



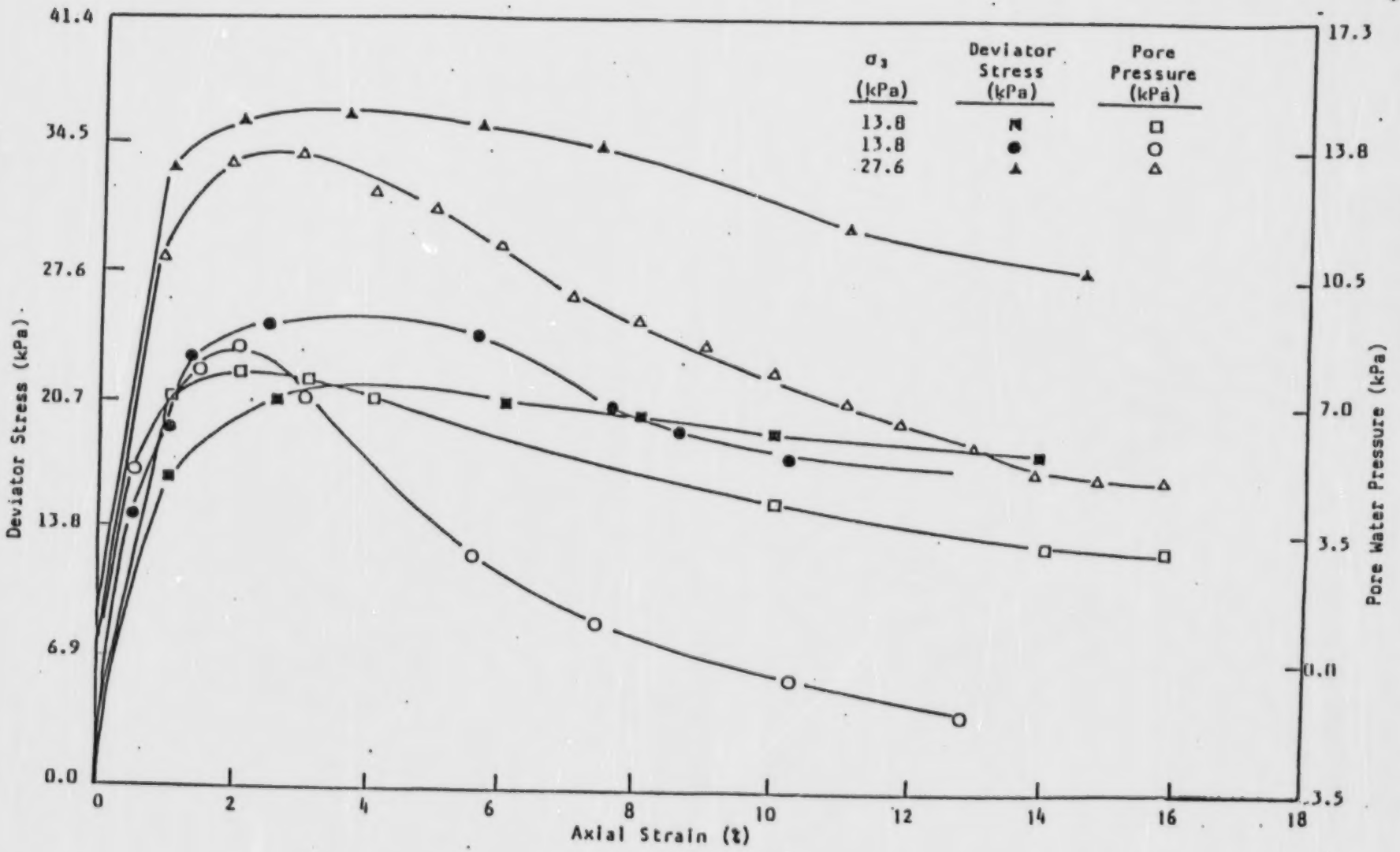


Fig. 11 Development of Deviator Stress and Pore Pressure with Axial Strain in Static Triaxial Tests on Composite Soil "B"

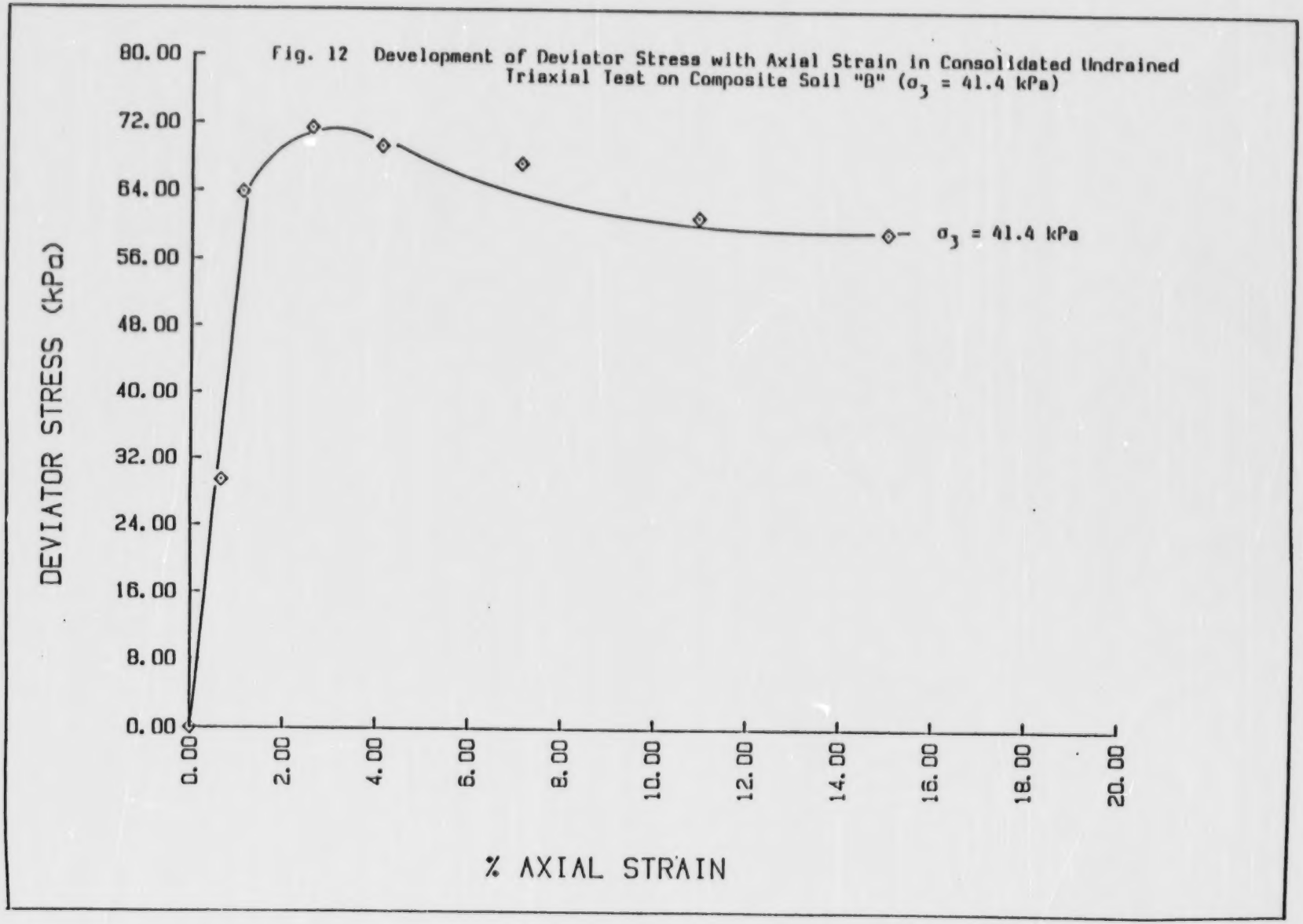


Fig. 12 Development of Deviator Stress with Axial Strain in Consolidated Undrained Triaxial Test on Composite Soil "B" (σ_3 = 41.4 kPa)
80.00
72.00
64.00
56.00
48.00
40.00
32.00
24.00
16.00
8.00
0.00
DEVIATOR STRESS (kPa)
σ_3 = 41.4 kPa
0.00
2.00
4.00
6.00
8.00
10.00
12.00
14.00
16.00
18.00
20.00
% AXIAL STRAIN

Fig. 13 Development of Pore Water Pressure with Axial Strain in Consolidated Undrained Triaxial Test on Composite Soil "B" (σ_3 = 41.4 kPa)

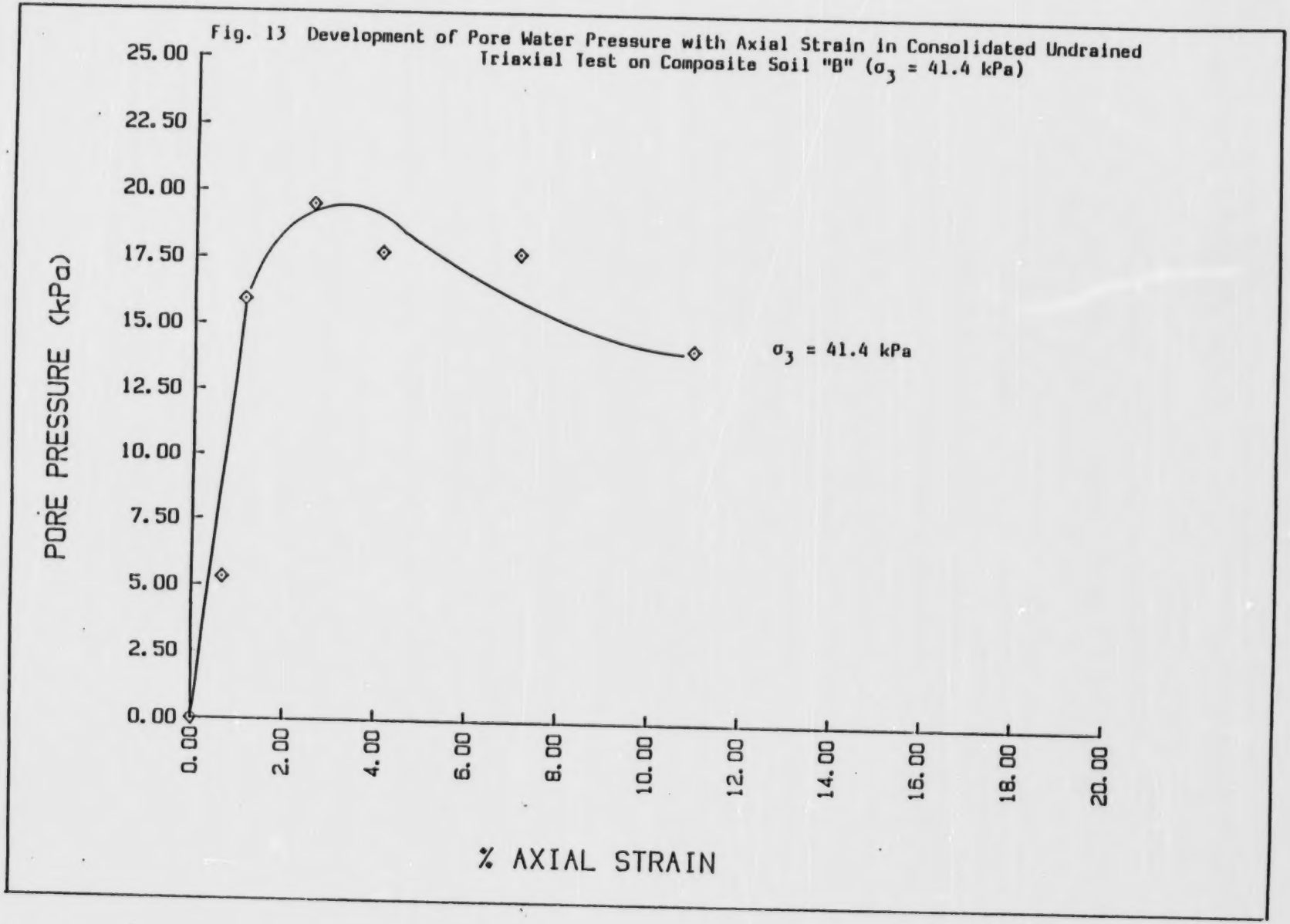


Fig. 14 Stress Path Diagram for Static Tests Performed on Joliette Soil

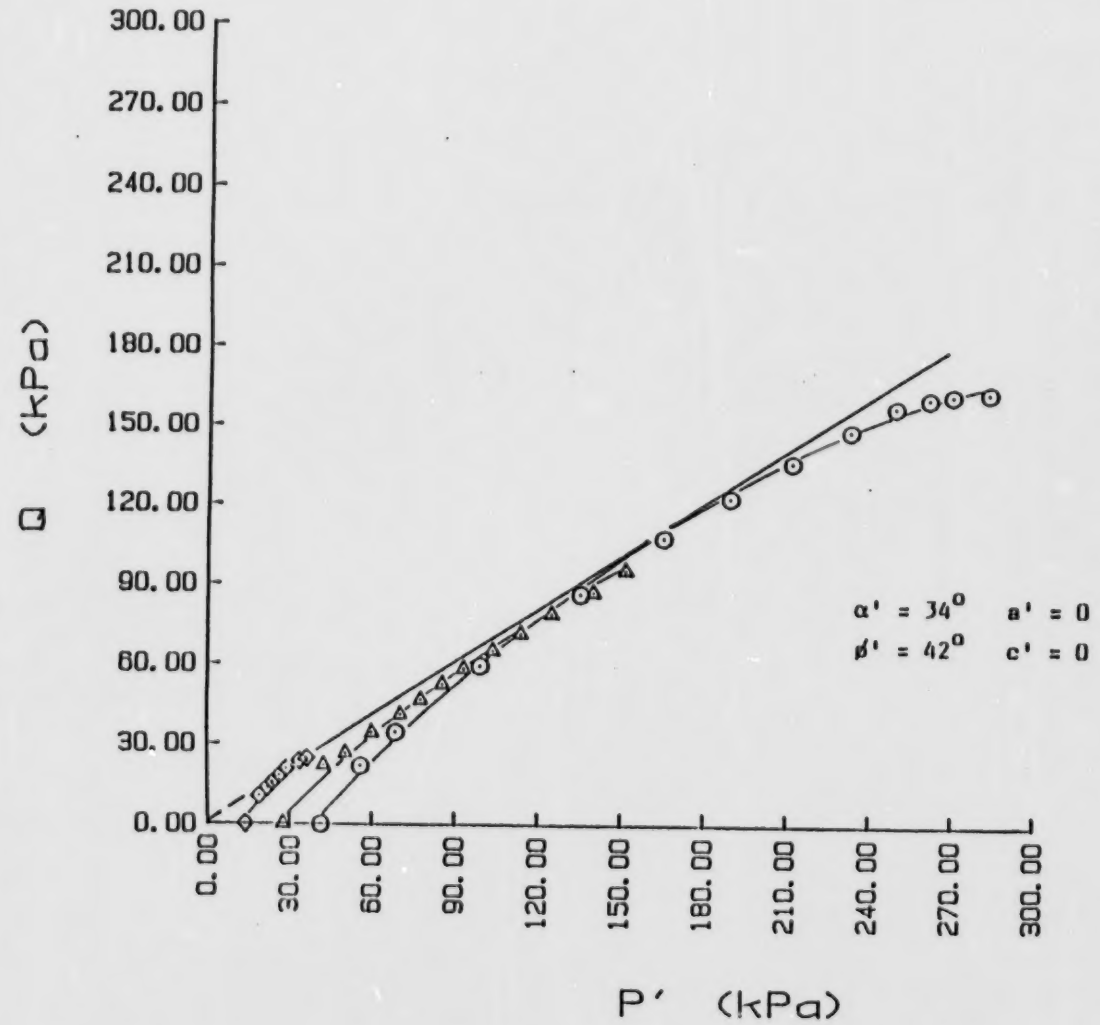
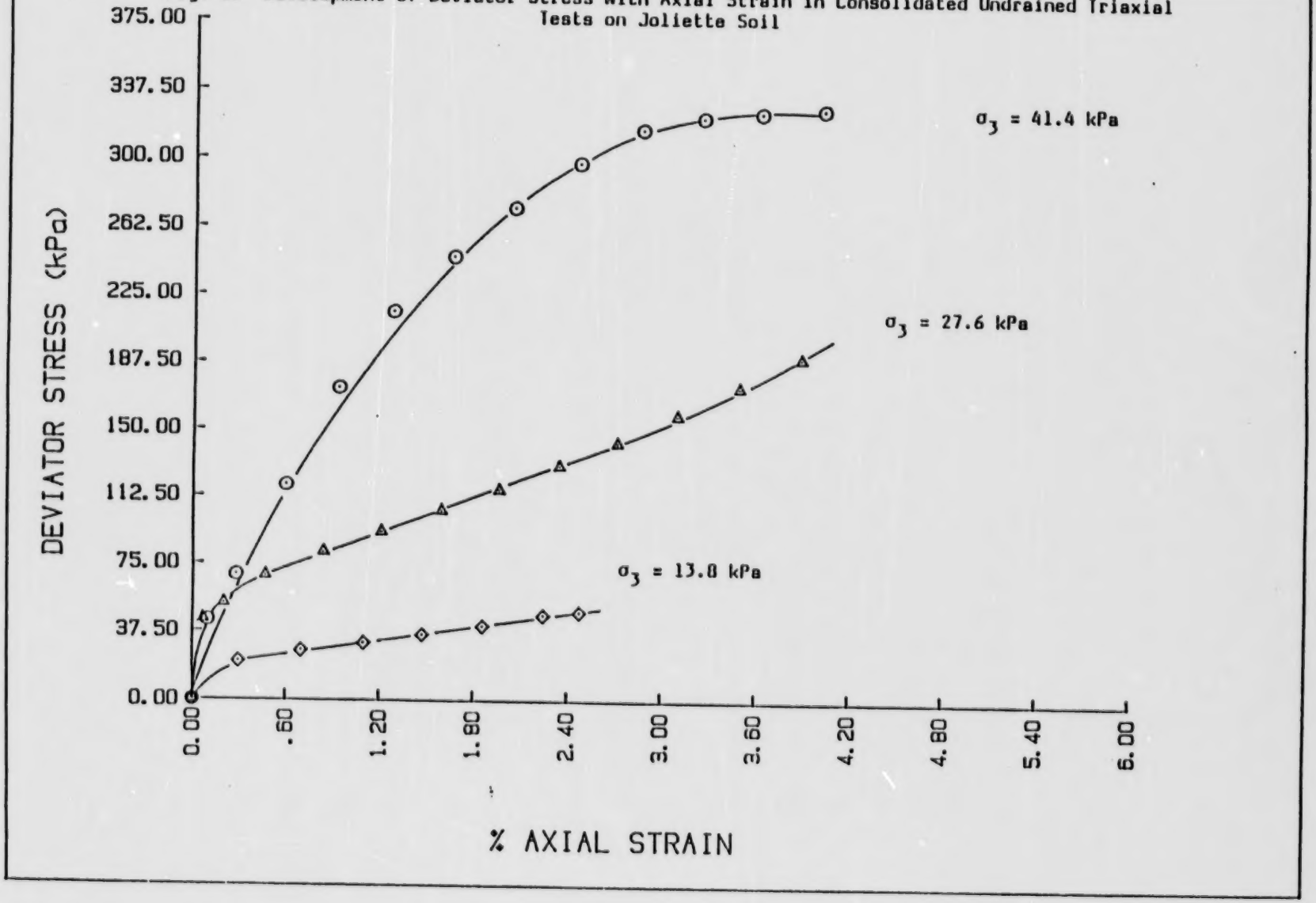


Fig. 15 Development of Deviator Stress with Axial Strain in Consolidated Undrained Triaxial Tests on Joliette Soil



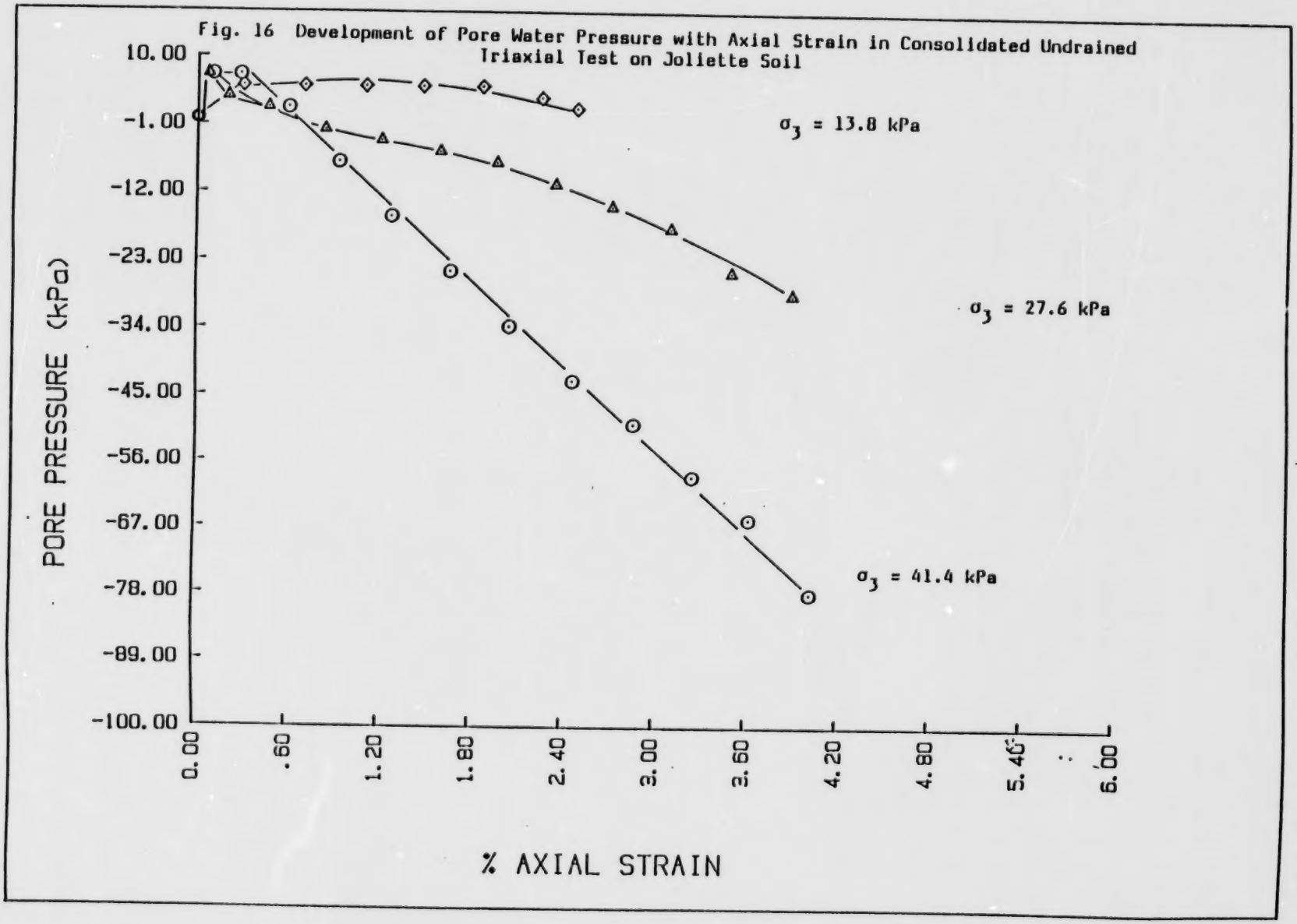


Fig. 16 Development of Pore Water Pressure with Axial Strain in Consolidated Undrained Triaxial Test on Joliette Soil

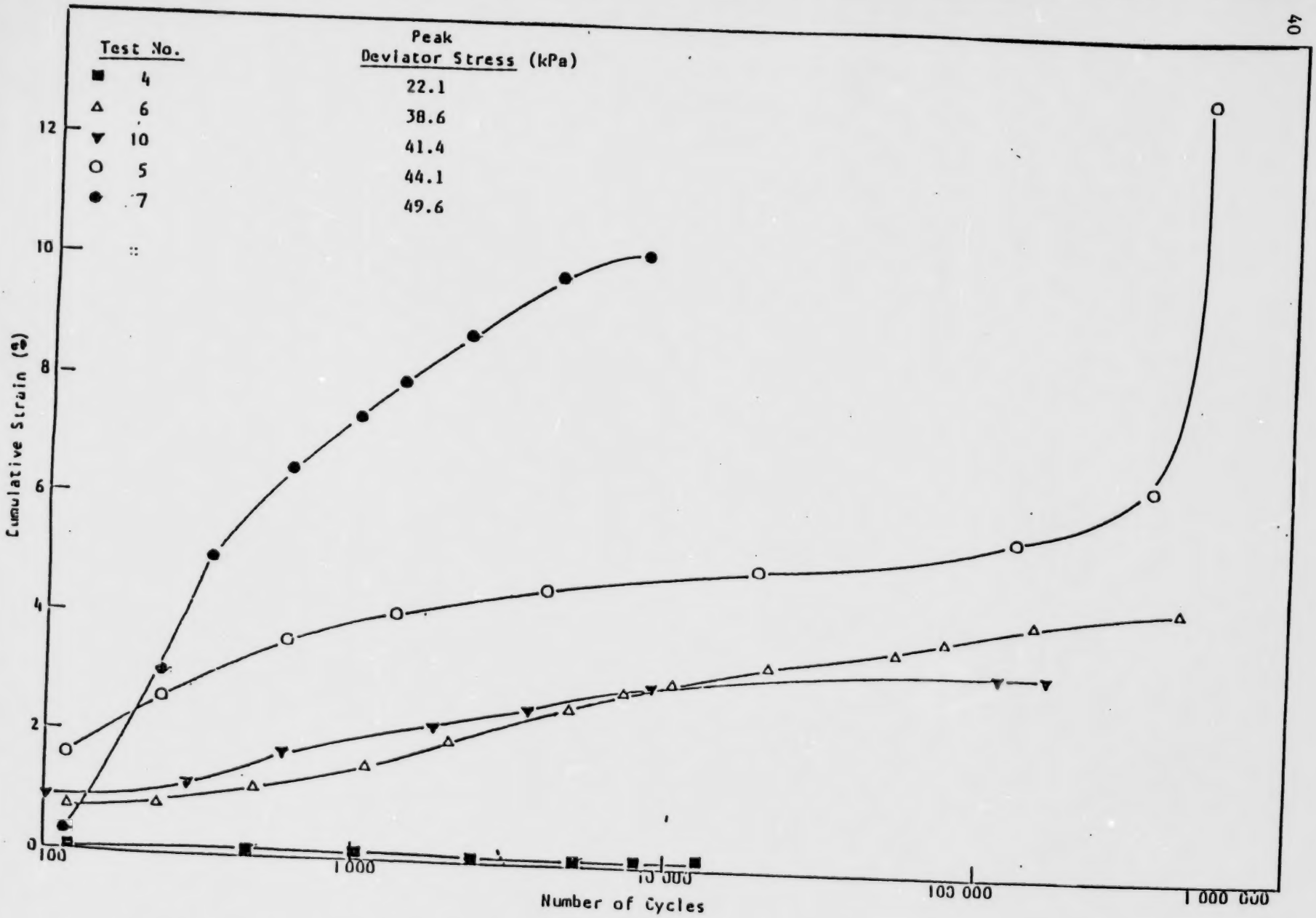


Fig. 17 RELATIONSHIPS BETWEEN CUMULATIVE AXIAL STRAIN AND NUMBER OF LOAD CYCLES FOR SPECIMENS FROM COMPOSITE SAMPLE "A" (σ_3 = 13.8kPa).

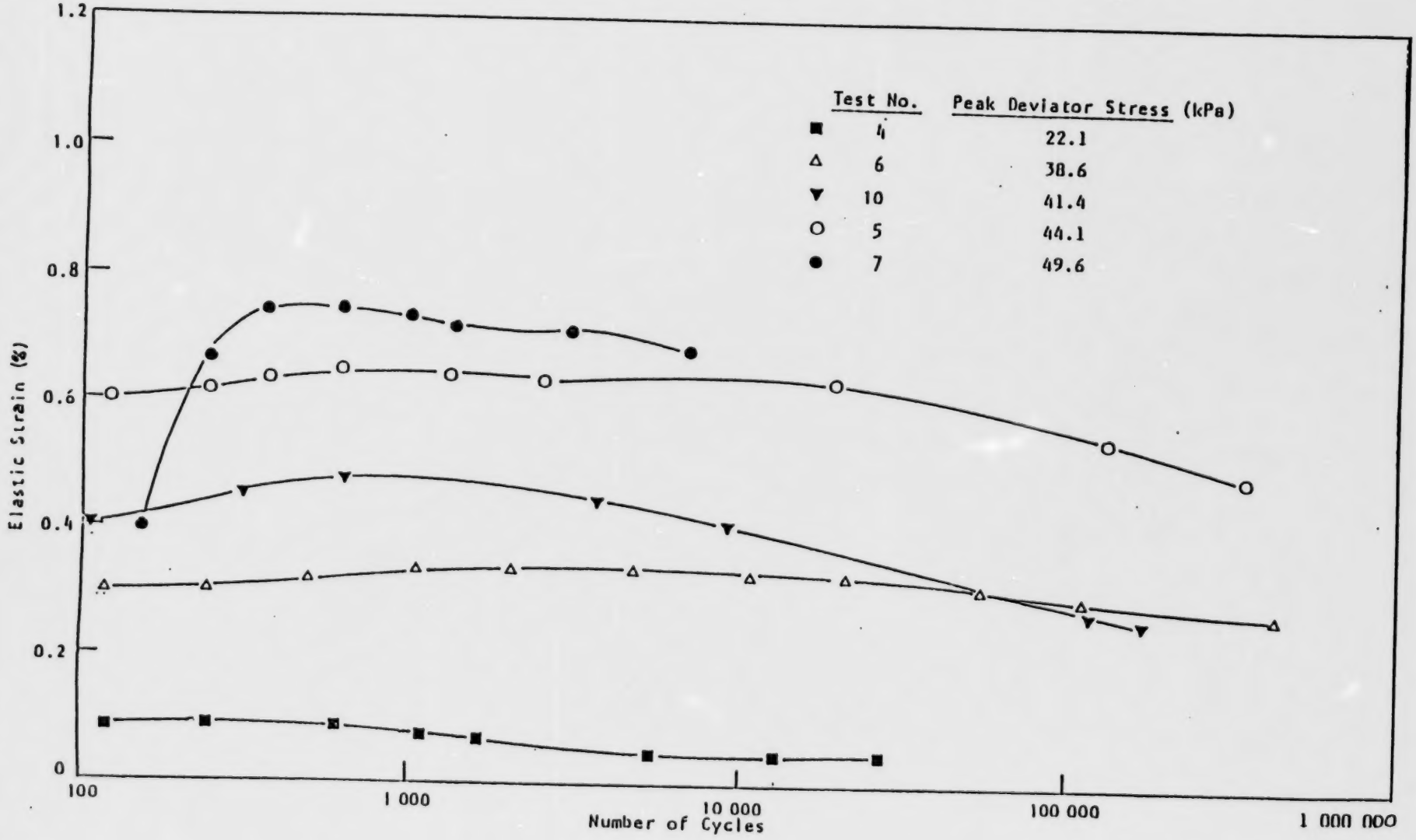


Fig. 18 RELATIONSHIPS BETWEEN AXIAL ELASTIC STRAIN AND NUMBER OF LOAD CYCLES FOR SPECIMENS FROM COMPOSITE SAMPLE "A" (σ_3 = 13.8 kPa).

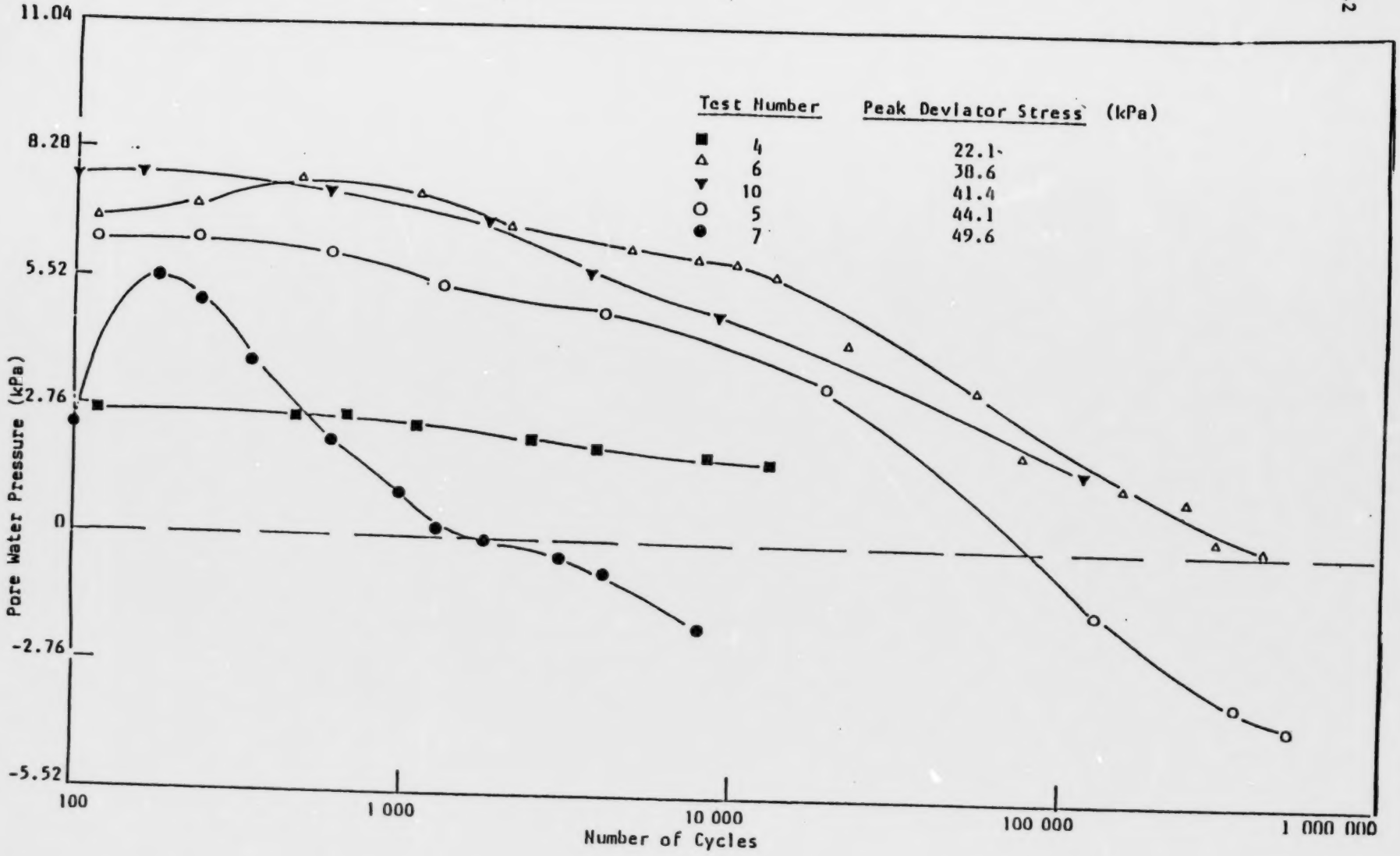


Fig. 19 VARIATION IN PORE PRESSURE WITH NUMBER OF LOAD CYCLES FOR SPECIMEN FROM COMPOSITE SAMPLE "A". (σ_3 = 13.8kPa)

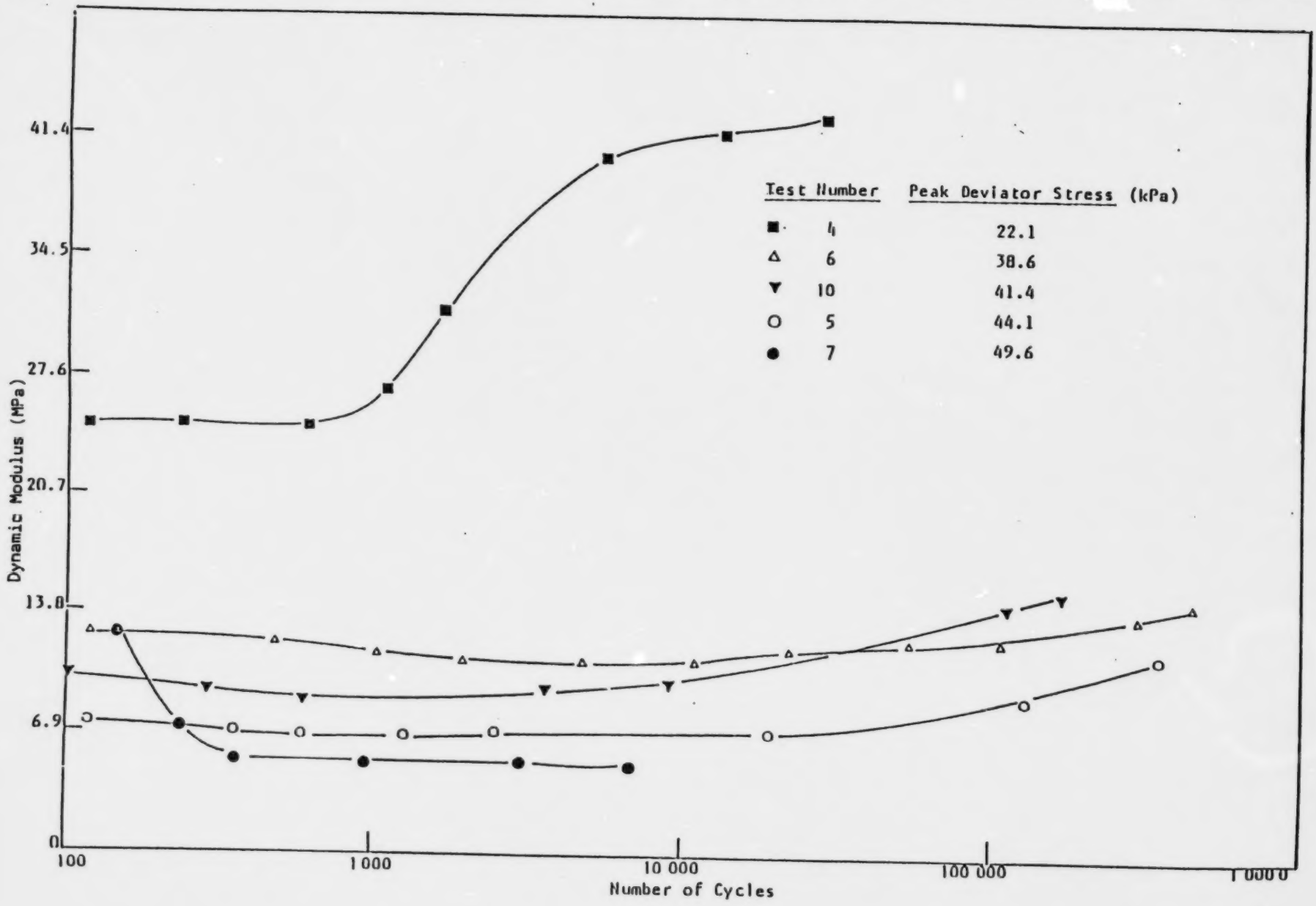


Fig. 20 VARIATION IN DYNAMIC MODULUS WITH NUMBER OF LOAD CYCLES FOR SPECIMENS FROM COMPOSITE SAMPLE "A".

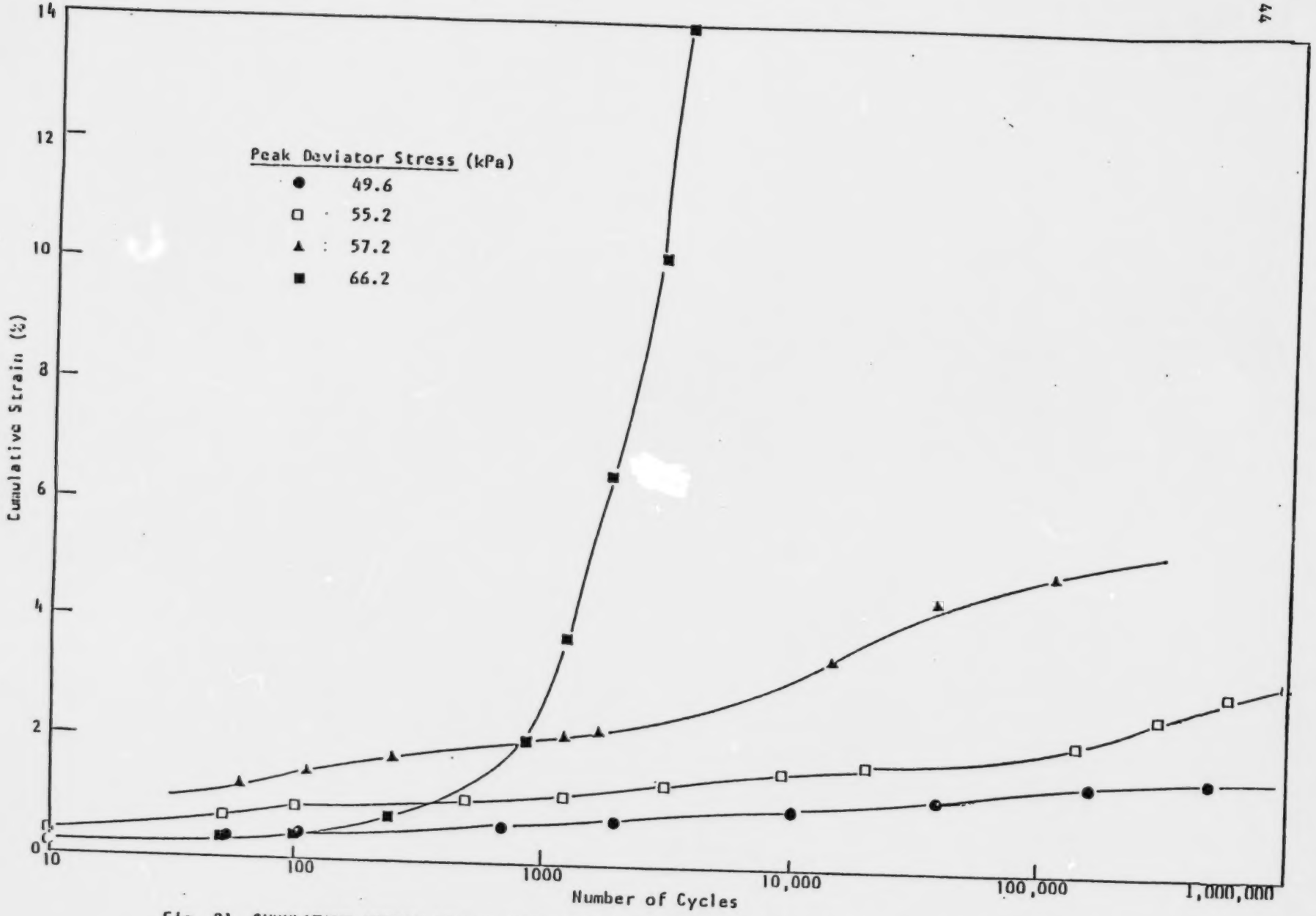


Fig. 21 CUMULATIVE STRAIN VERSUS NUMBER OF LOAD CYCLES FOR COMPOSITE SOIL "A" (σ_3 = 27.6 kPa)

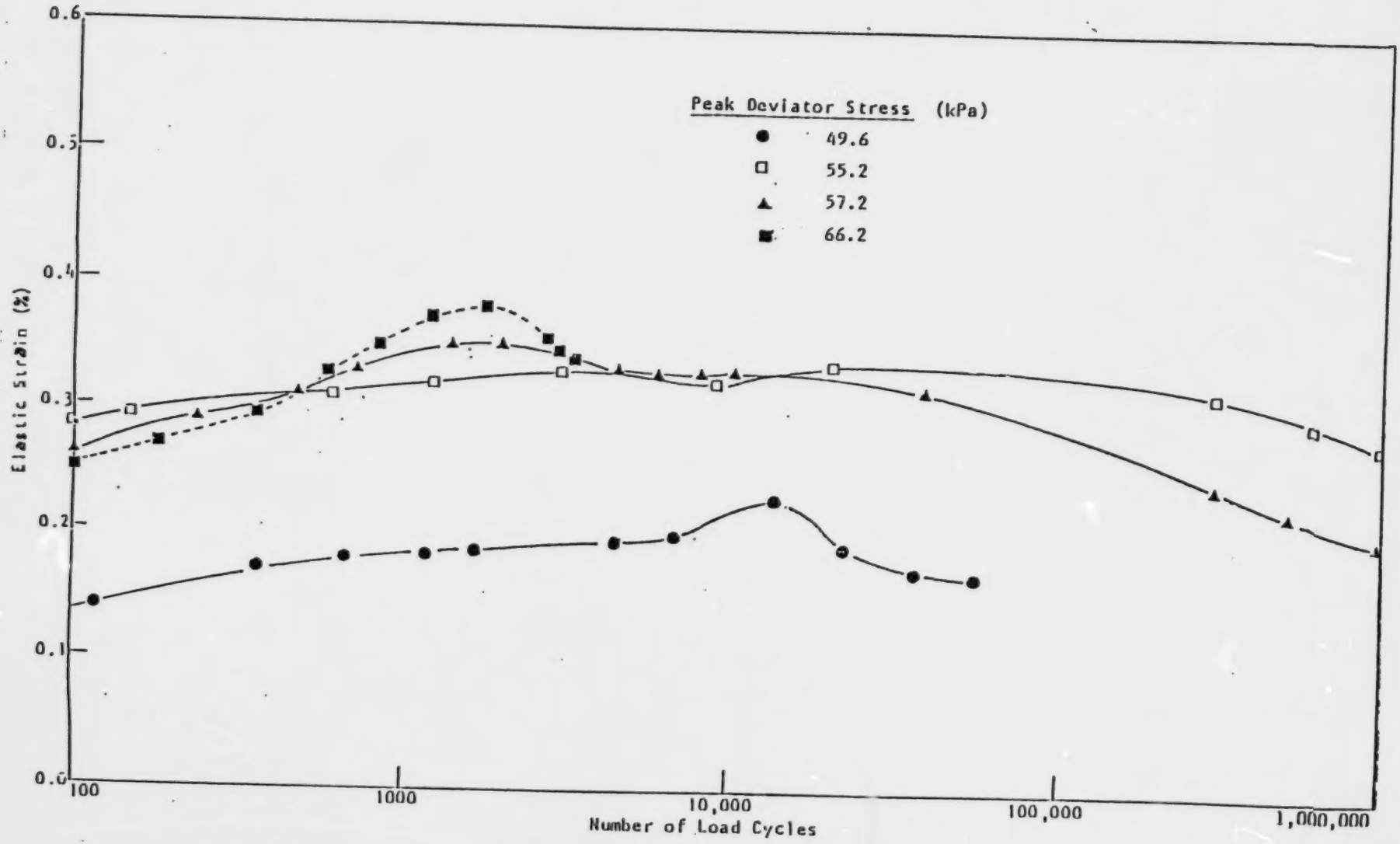


Fig. 22 ELASTIC STRAIN VERSUS NUMBER OF LOAD CYCLES FOR COMPOSITE SOIL "A" (σ_3 = 27.6 kPa)

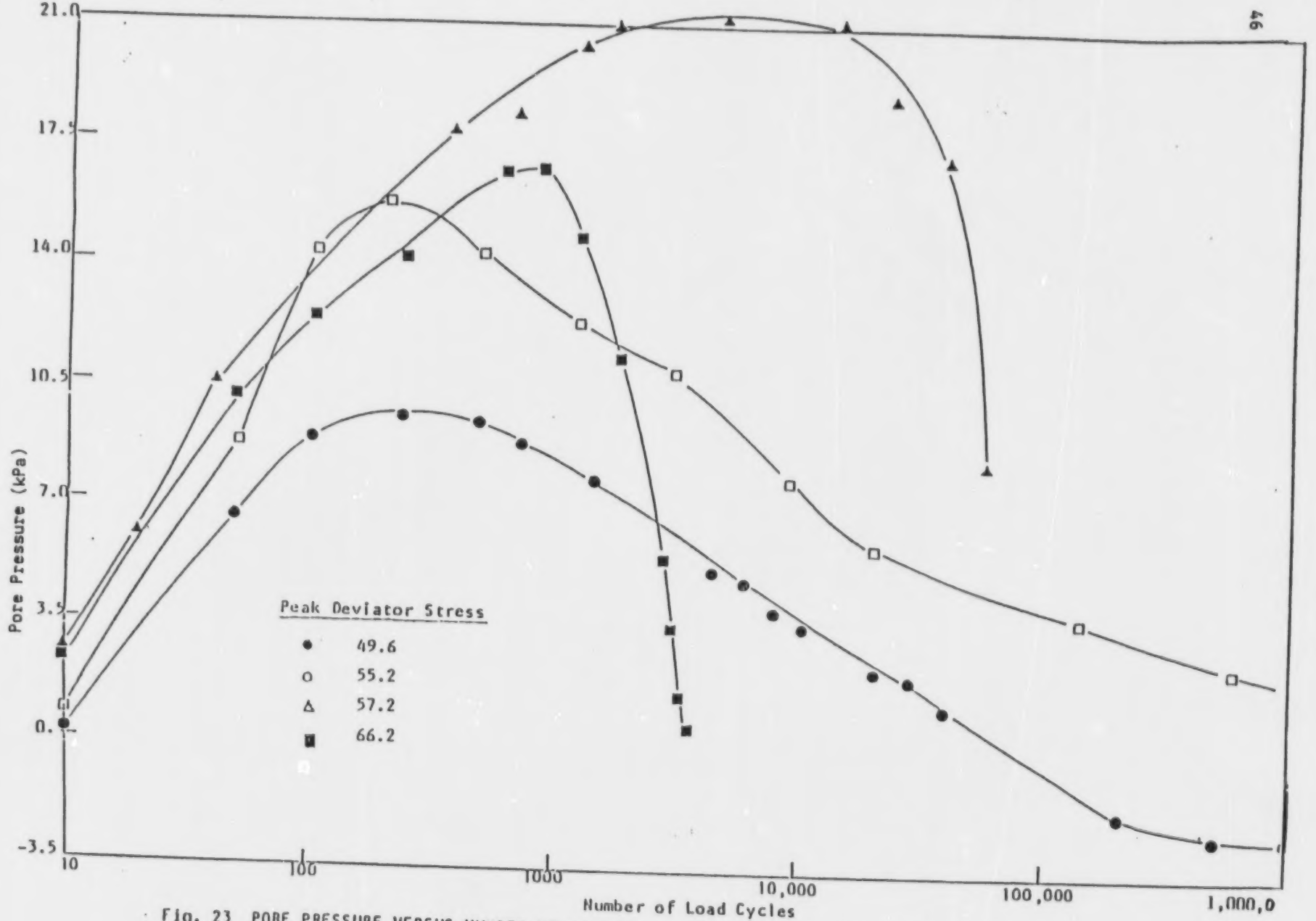


Fig. 23 PORE PRESSURE VERSUS NUMBER OF LOAD CYCLES FOR COMPOSITE SOIL [illegible]

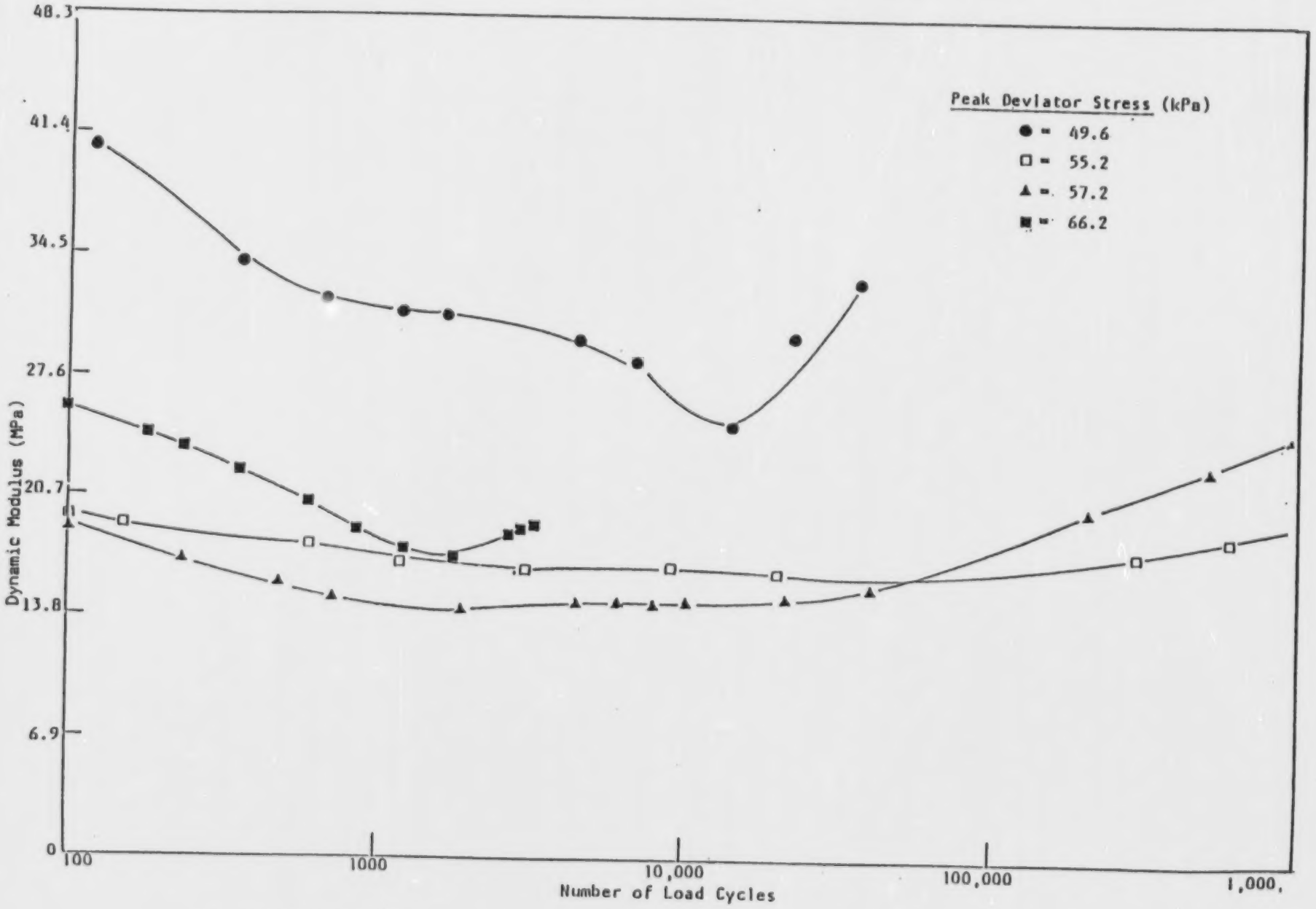


Fig. 24 DYNAMIC MODULUS VERSUS NUMBER OF LOAD CYCLES FOR COMPOSITE SOIL "A" (σ_3 = 27.6 kPa)

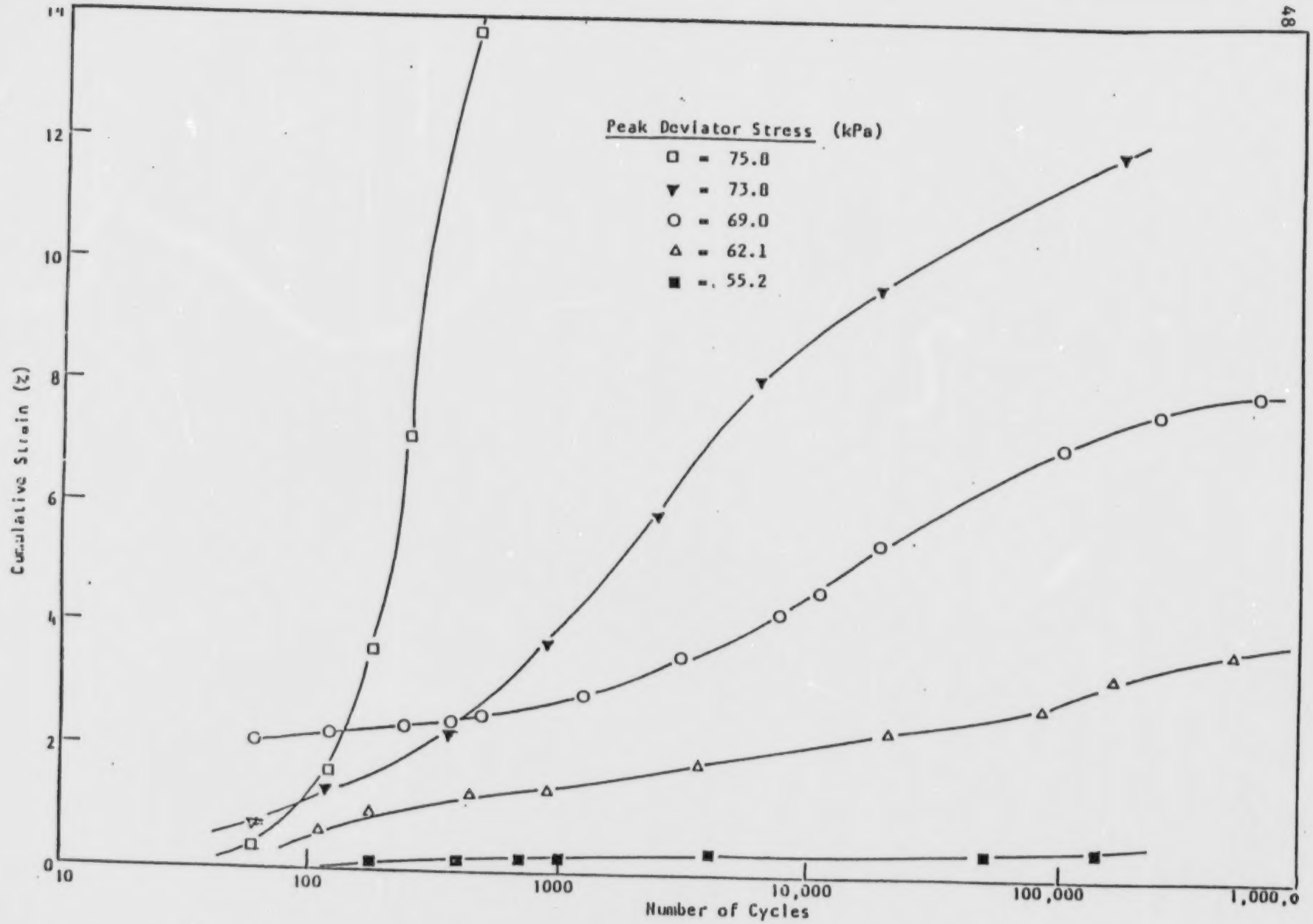


Fig. 25 CUMULATIVE STRAIN VERSUS NUMBER OF LOAD CYCLES FOR COMPOSITE SOIL "A" (σ_3 = 41.4 kPa)

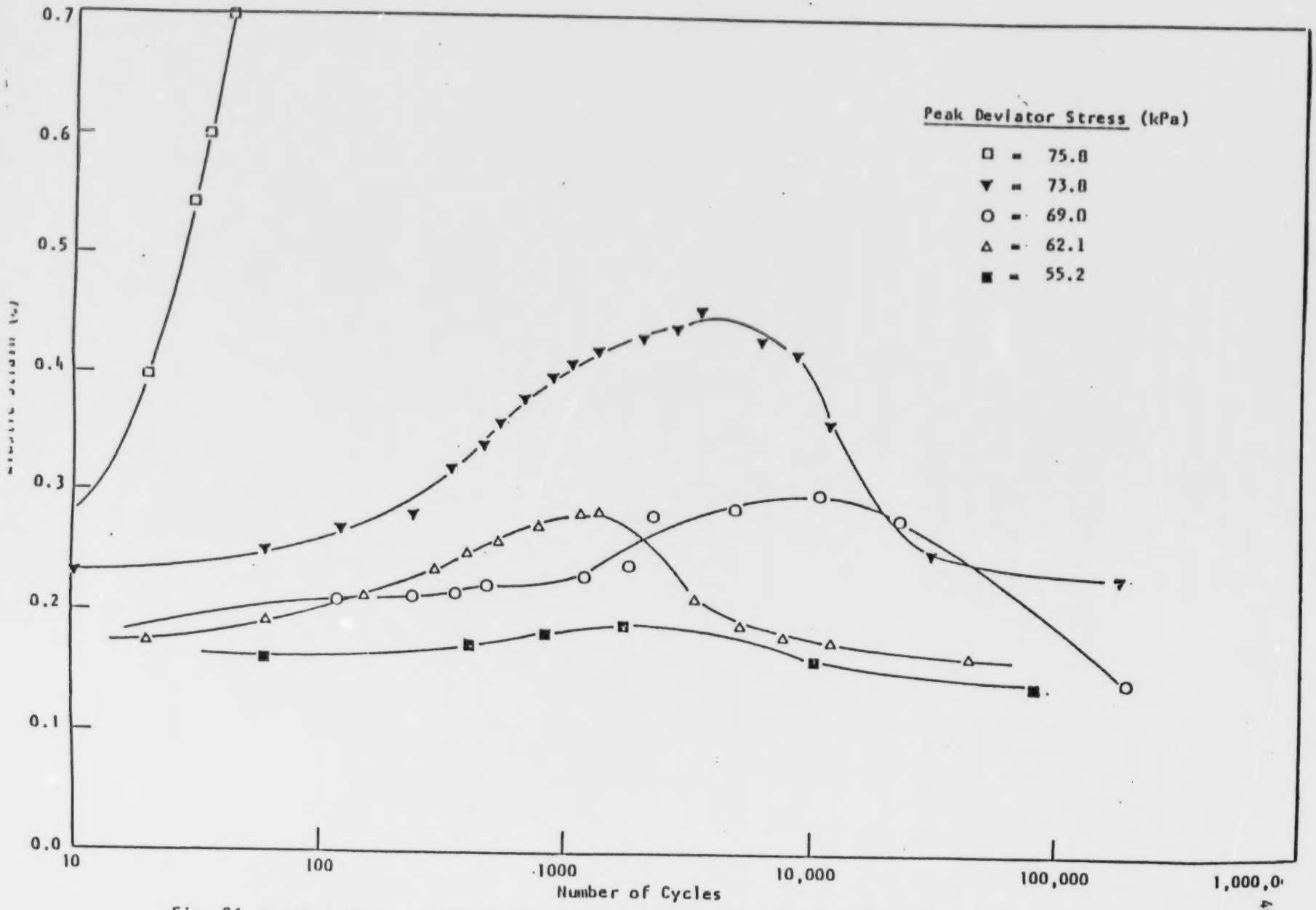


Fig. 26 ELASTIC STRAIN VERSUS NUMBER OF LOAD CYCLES FOR COMPOSITE SOIL "A" (σ_3 = 41.4 kPa)

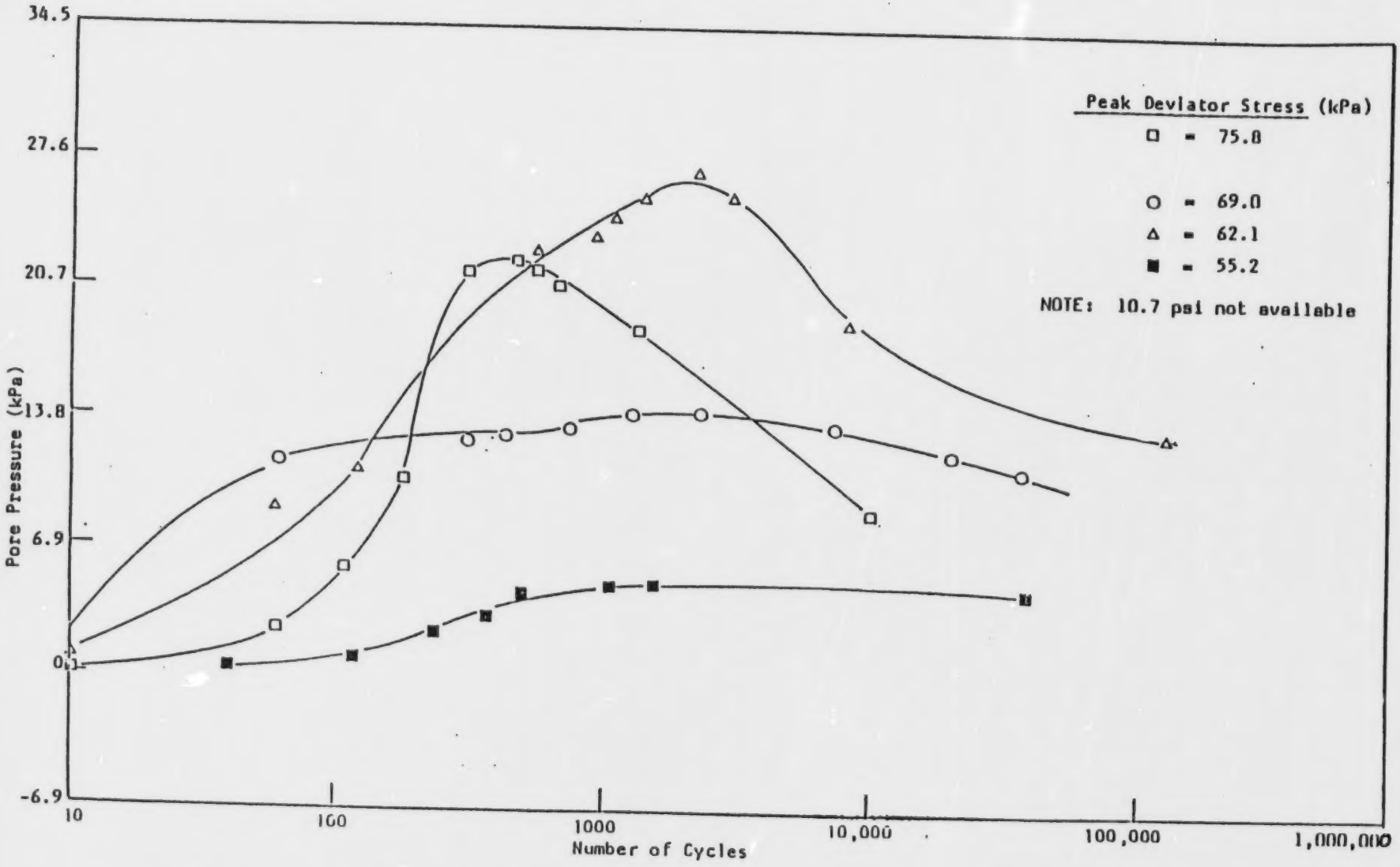


Fig. 27 PORE PRESSURE VERSUS NUMBER OF LOAD CYCLES FOR COMPOSITE SOIL "A" (σ_3 = 41.4 kPa)

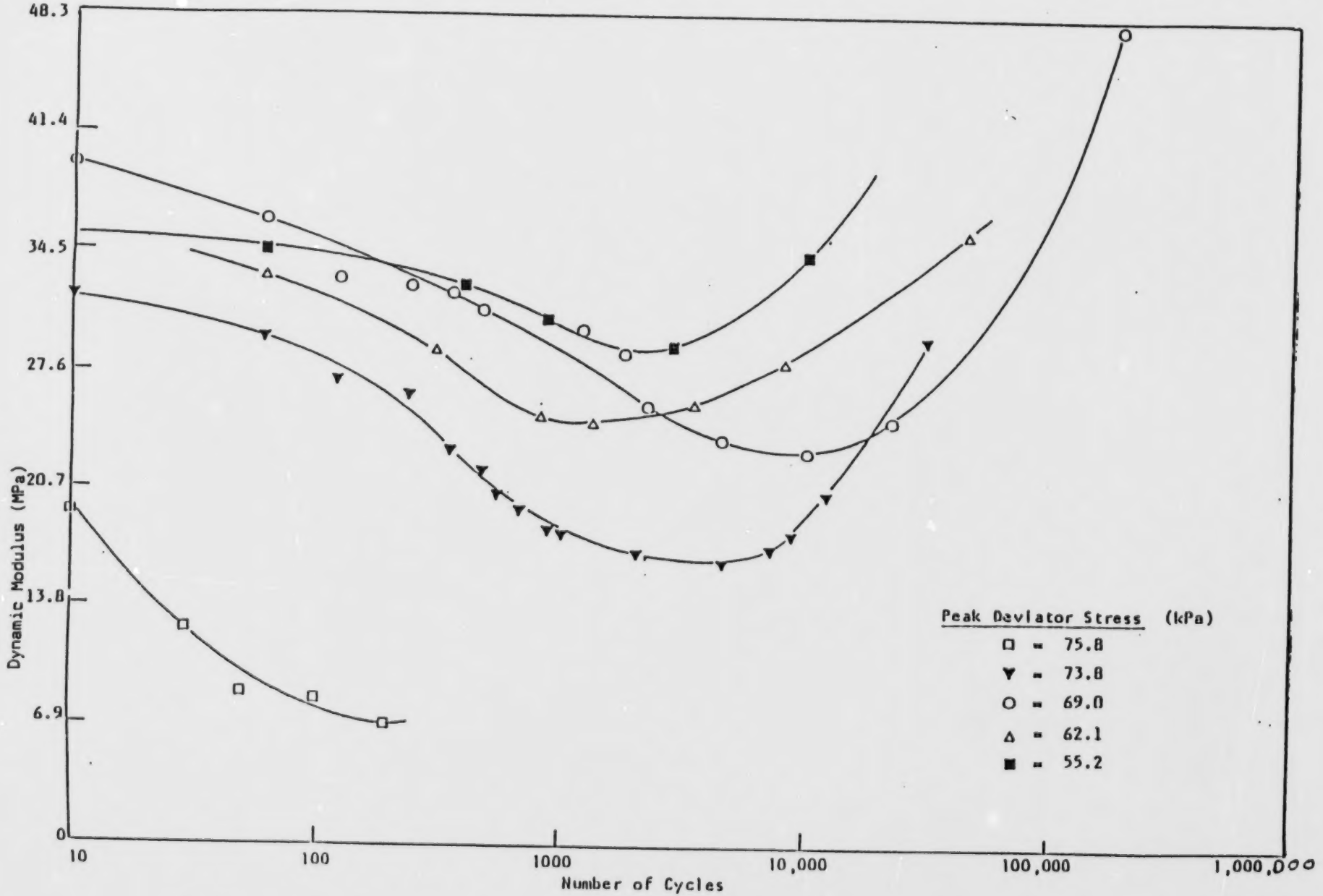


Fig. 28 DYNAMIC MODULUS VERSUS NUMBER OF LOAD CYCLES FOR COMPOSITE SOIL "A" (σ_3 = 41.4 kPa)

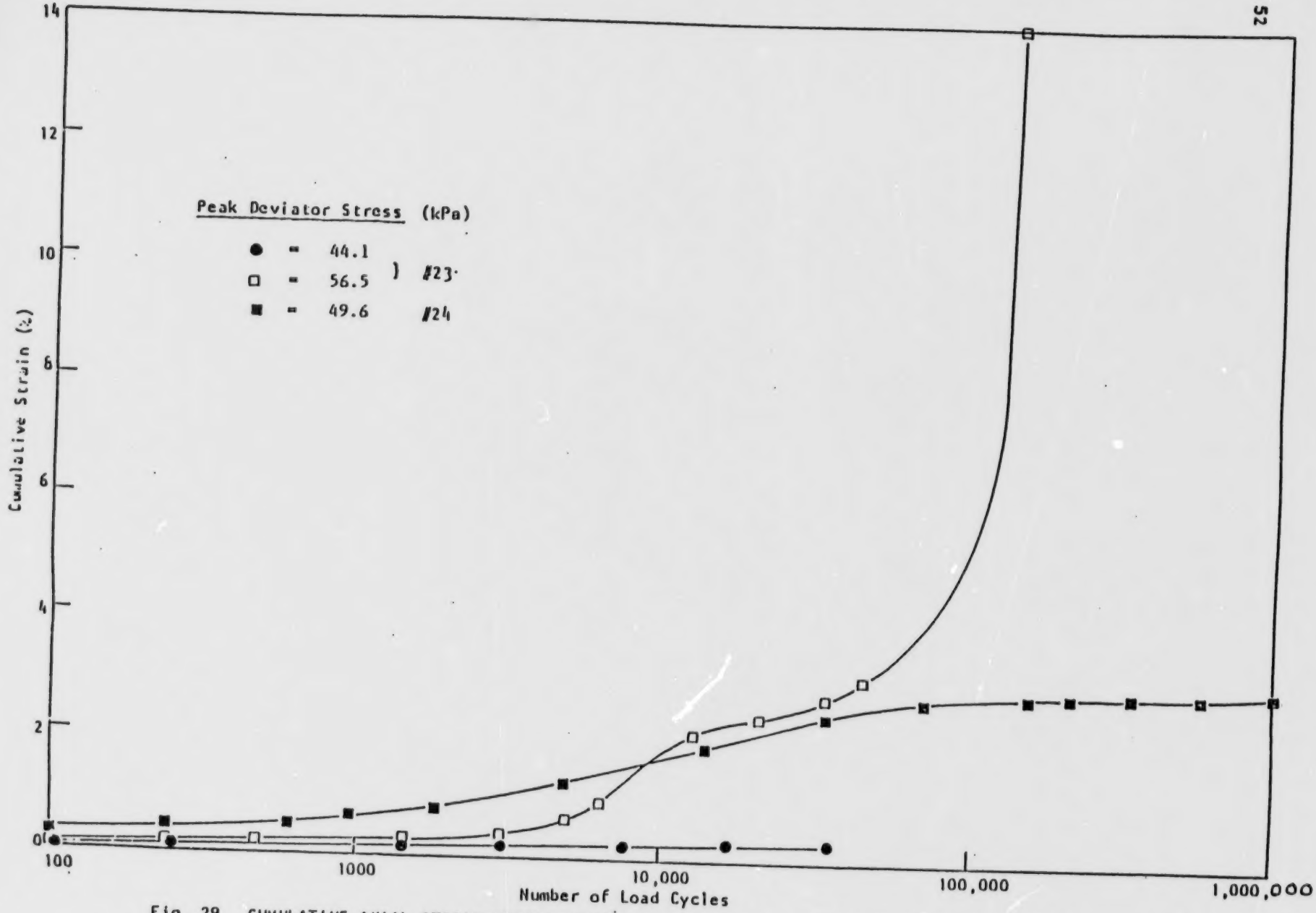


Fig. 29 CUMULATIVE AXIAL STRAIN VERSUS NUMBER OF LOAD CYCLES FOR SPECIMENS FROM COMPOSITE SAMPLE "A" UNDER DRAINED TEST CONDITIONS. (σ_3 = 13.8 kPa)

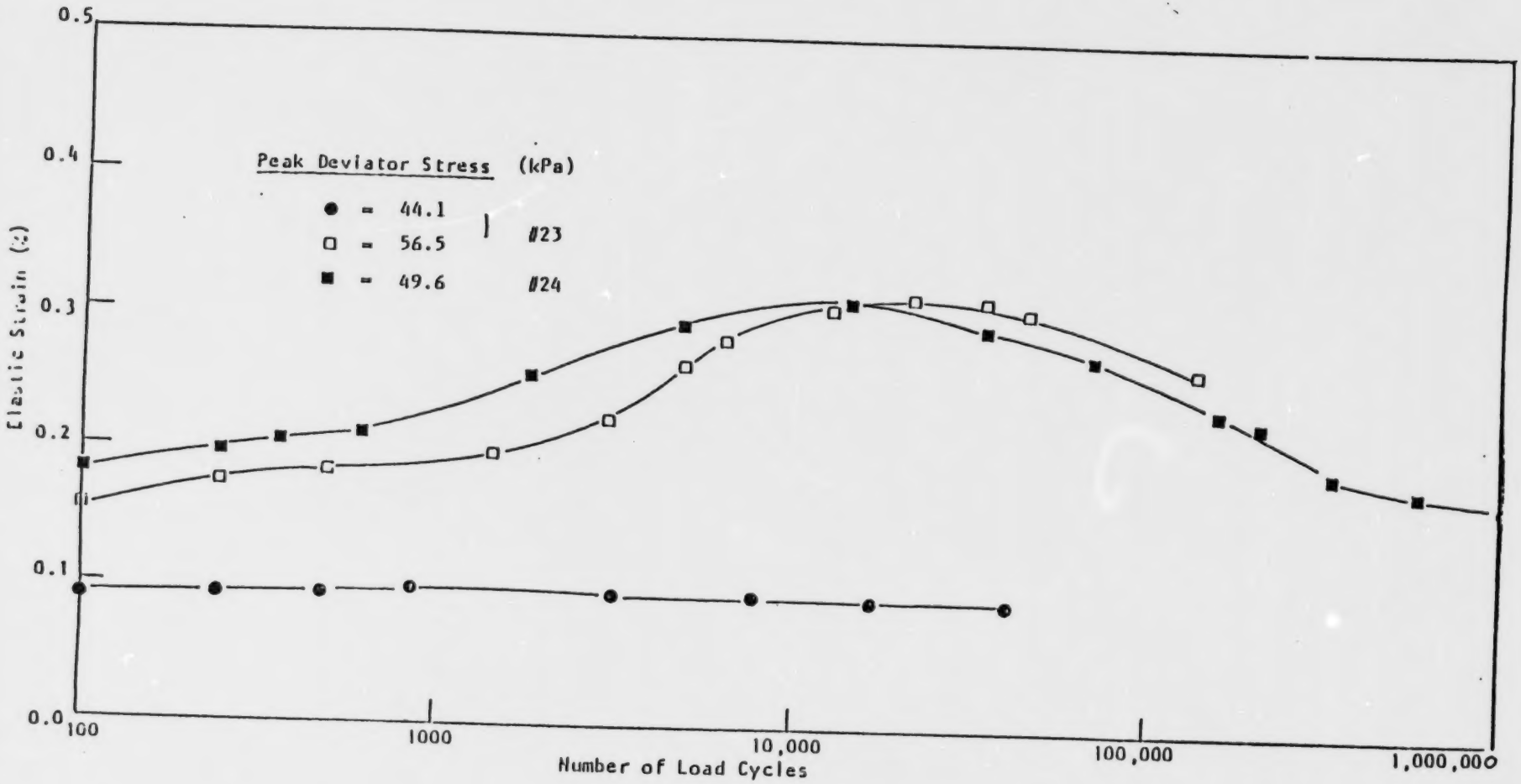
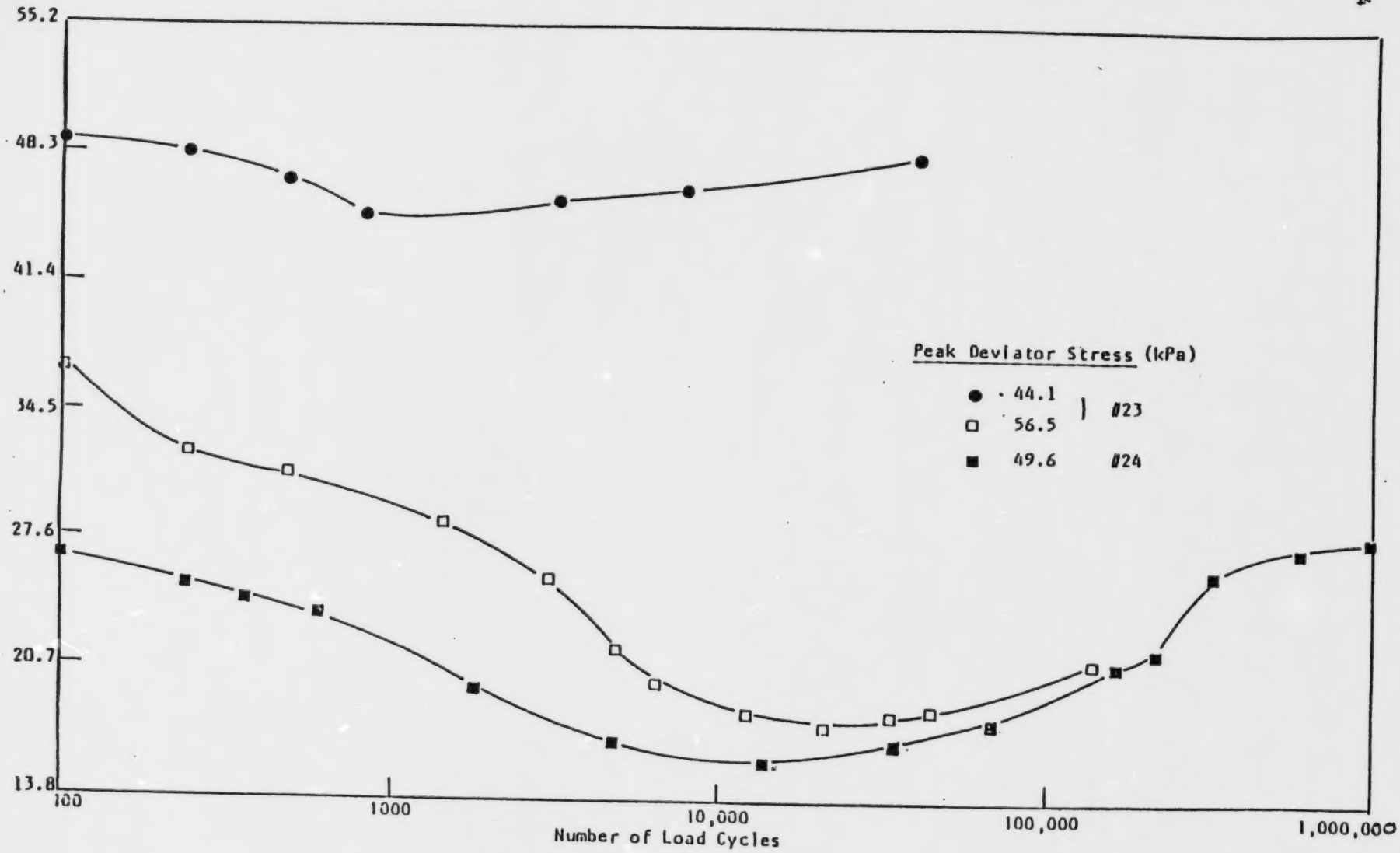


Fig. 30 ELASTIC STRAIN VERSUS NUMBER OF LOAD CYCLES FOR SOIL "A" UNDER DRAINED TEST CONDITIONS. (σ_3 = 13.8 kPa)



Fgi. 31 DYNAMIC MODULUS VERSUS NUMBER OF LOAD CYCLES FOR SOIL "A" UNDER DRAINED TEST CONDITIONS. (σ_3 = 13.8 kPa)

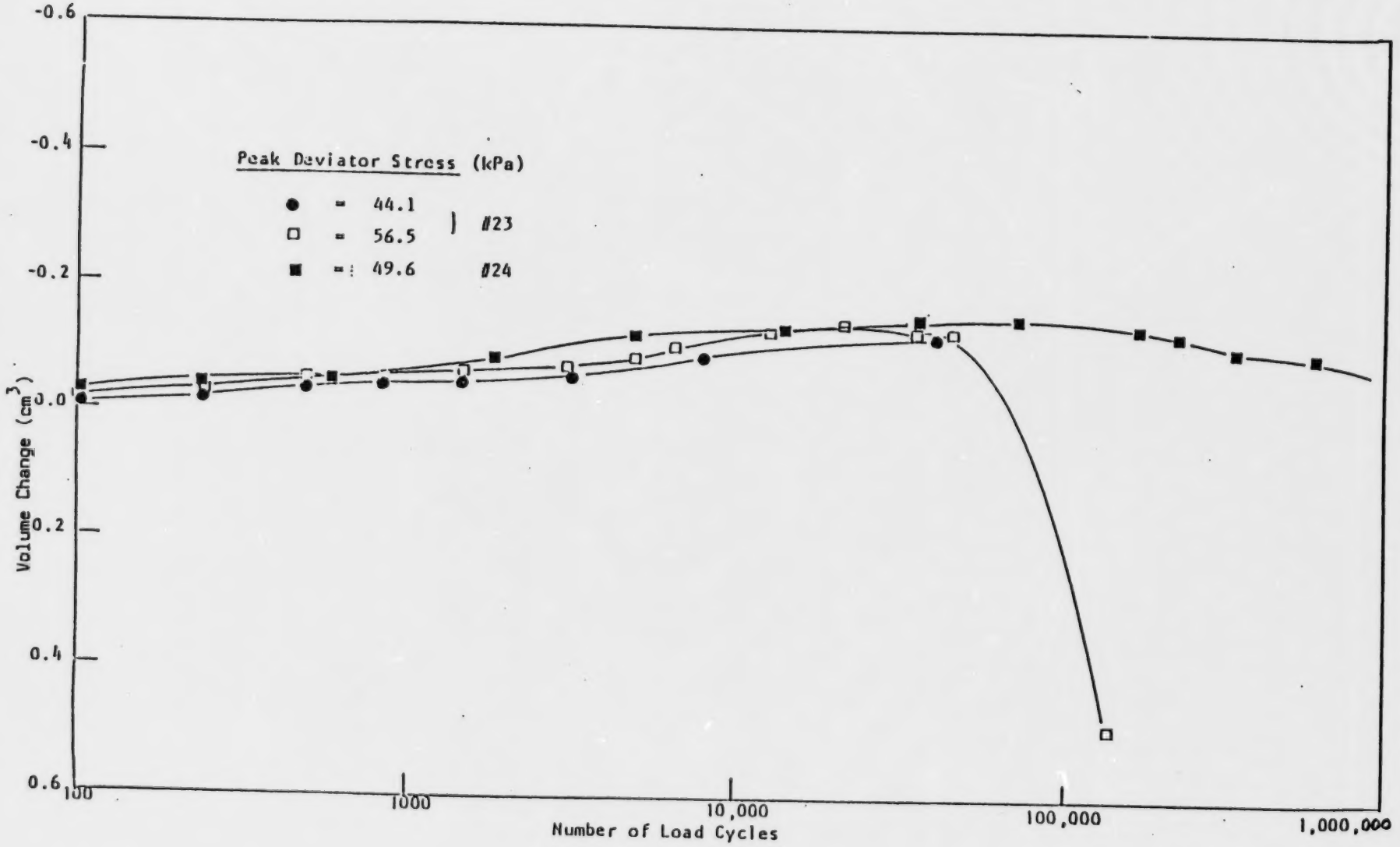


Fig. 32 VARIATION OF VOLUME WITH THE NUMBER OF LOAD CYCLES FOR COMPOSITE SOIL "A" UNDER DRAINED TEST CONDITIONS. (σ_3 = 13.8 kPa)

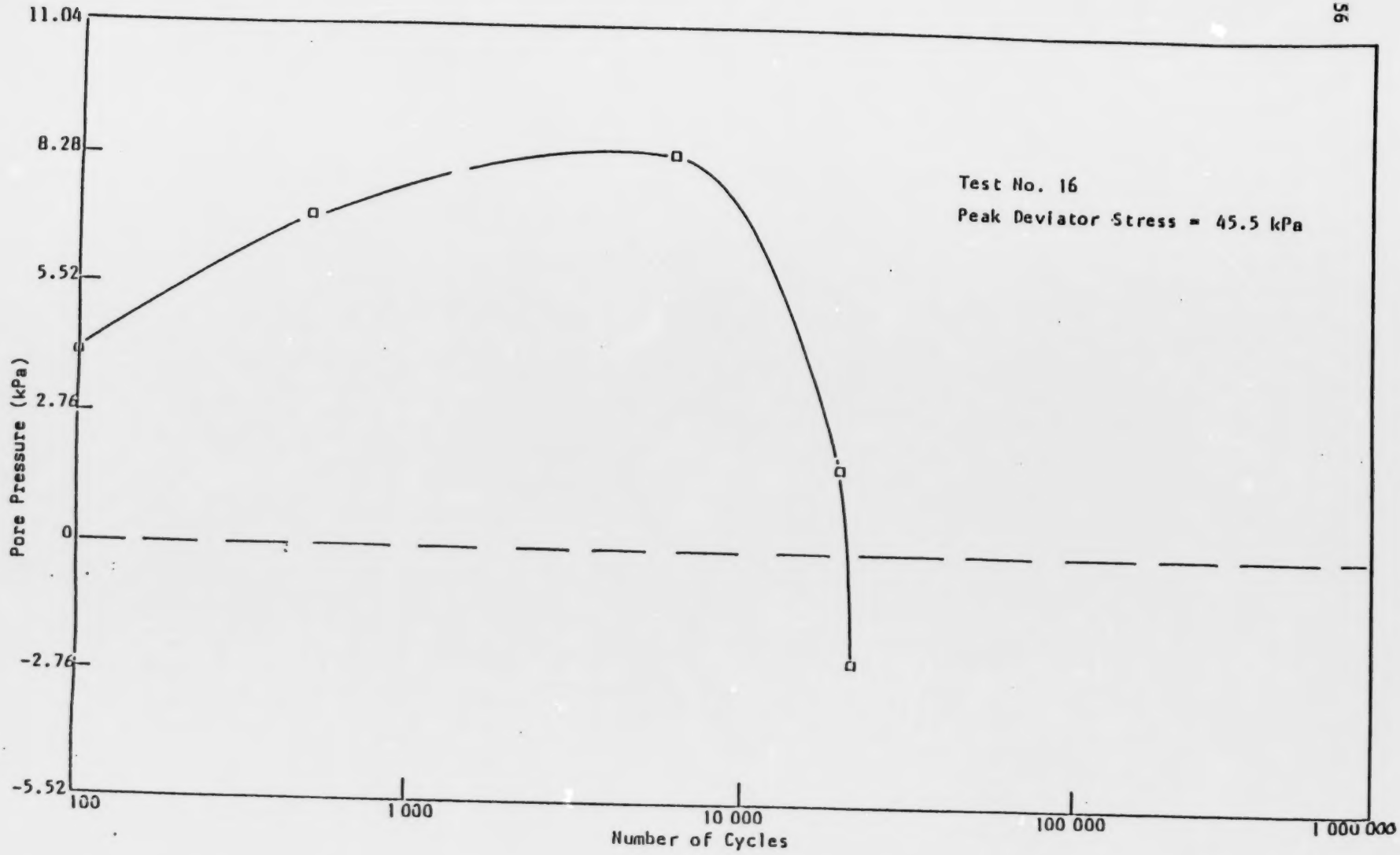


Fig. 33 VARIATION IN PORE PRESSURE WITH NUMBER OF LOAD CYCLES FOR A PERIODICALLY DRAINED TEST ON COMPOSITE SOIL "A".

Test Number	Soil Type	Test Condition	Consolidation Pressure (kPa)	Back Pressure (kPa)	Peak Cyclic Deviator Stress (kPa)	No. of Cycles	Cumulative Strain (%)	Range of Elastic Strain (%)	Range of Dynamic Modulus (MPa)	Range of Pore Pressure (kPa)	Initial Water Content (%)	Final Water Content (%)	Dry Density (Mg/m^3)	Remarks
12	B	undrained	13.8	241.3	45.5	1500	13.8	0.45-0.39	10.0-11.7	3.30-0.69	28.4	37.5	1.45	failed
14	B	"	13.8	275.8	45.5	6200	1.6	0.22-0.27	20.7-16.9	1.38-4.96				
					49.6	1300	13.2	0.45-0.38	10.7-13.1	2.76-3.86	28.5	36.3	1.47	failed
17	B	"	13.8	275.8	38.6	1000000	1.37	0.22-0.186	21.0-17.2	8.96-0	28.4	36.6	1.47	not failed
20	B	"	13.8	275.8	41.4	2200	14.0	0.34-0.24	16.9-12.1	9.38-2.07	27.4	36.0	1.49	failed

TABLE 6 Dynamic Triaxial Test Parameter for Specimens Prepared from Composite Sample "B" (Sine Pulse)

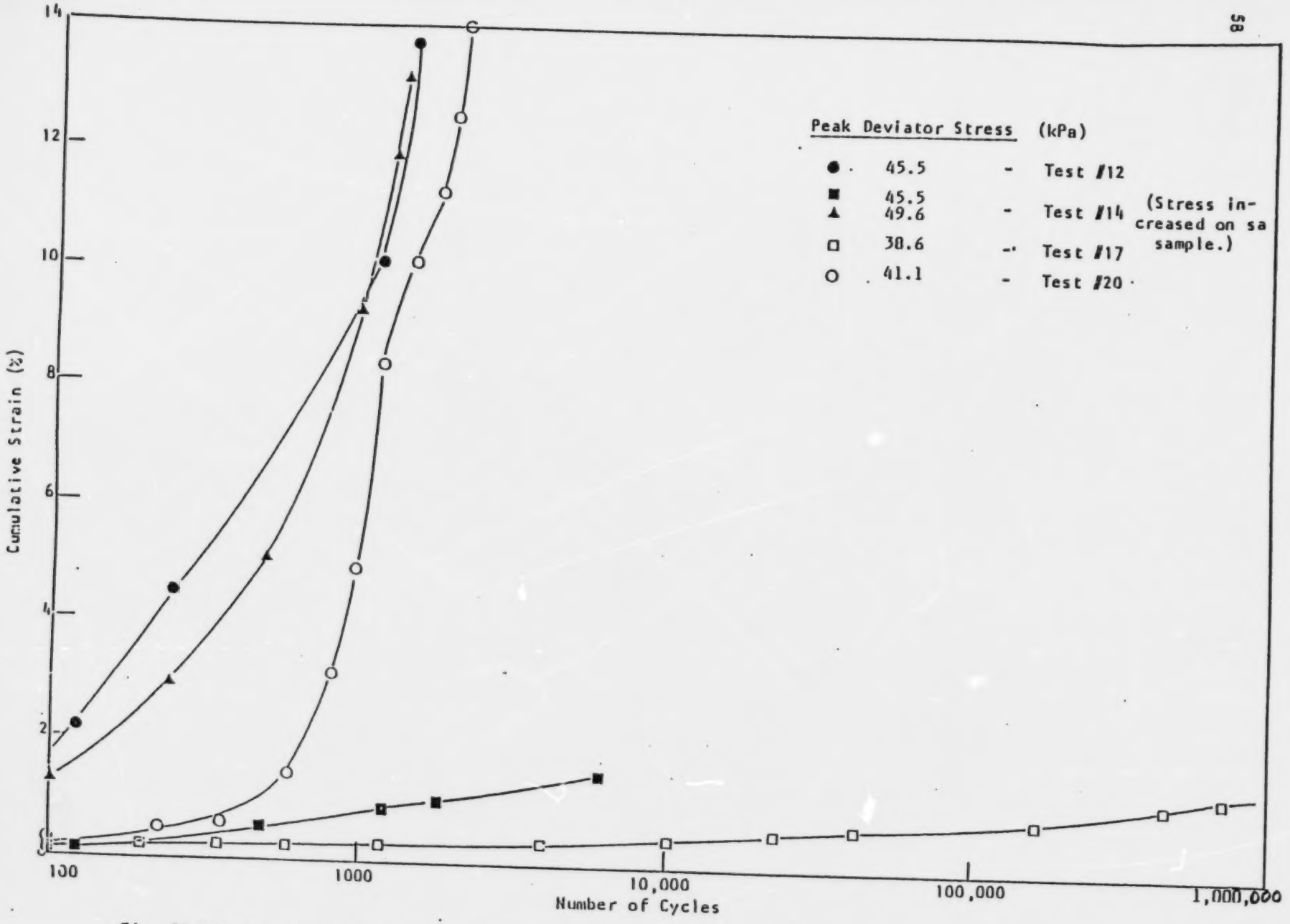


Fig. 34 RELATIONSHIPS BETWEEN CUMULATIVE AXIAL STRAIN AND NUMBER OF LOAD CYCLES FOR SPECIMENS FROM COMPOSITE SAMPLE "BB" (σ_3 = 13.8 kPa)

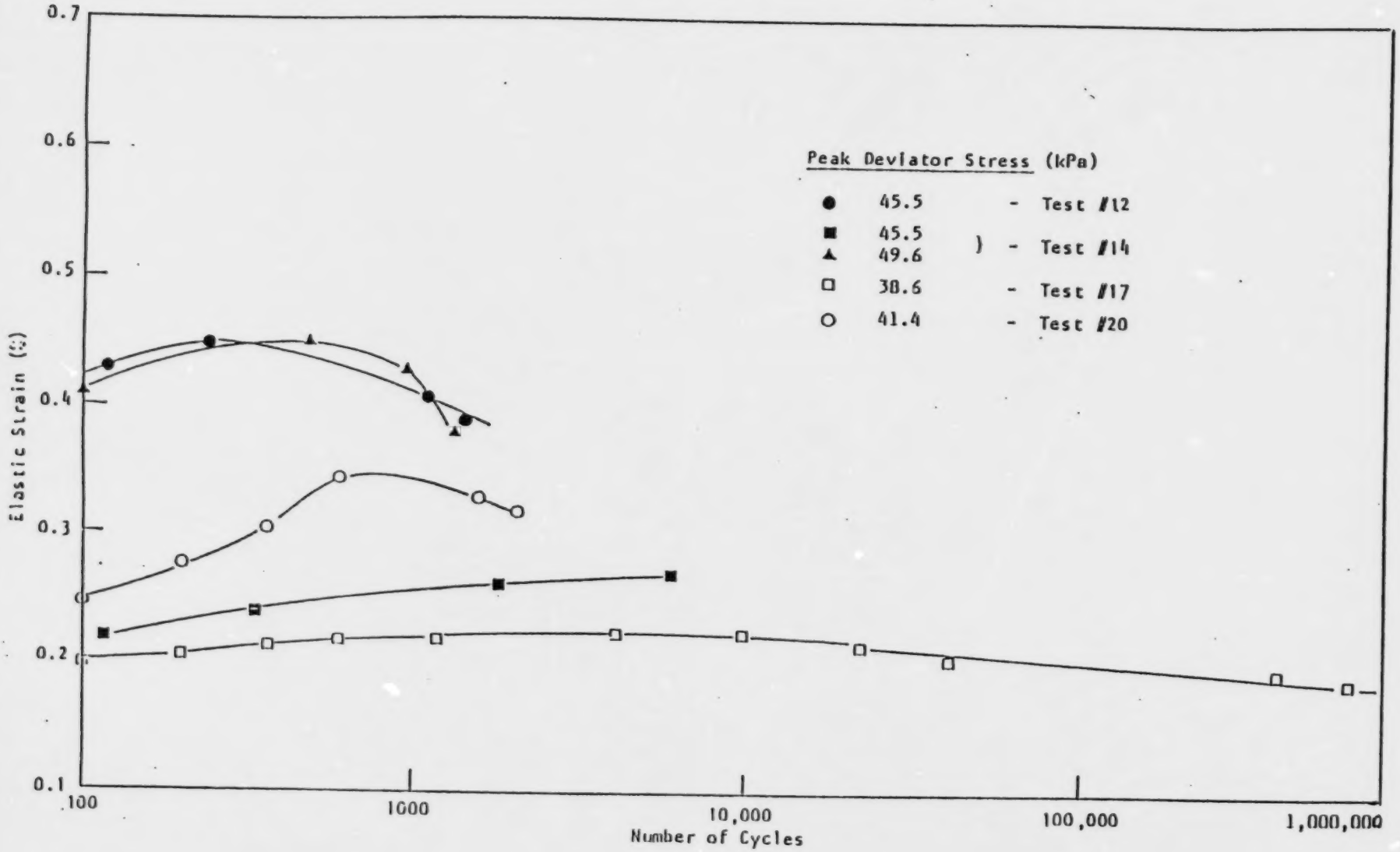


Fig. 35 RELATIONSHIPS BETWEEN AXIAL ELASTIC STRAIN AND NUMBER OF LOAD CYCLES FOR SPECIMENS FROM COMPOSITE SAMPLE "B". (σ_3 = 13.8 kPa)

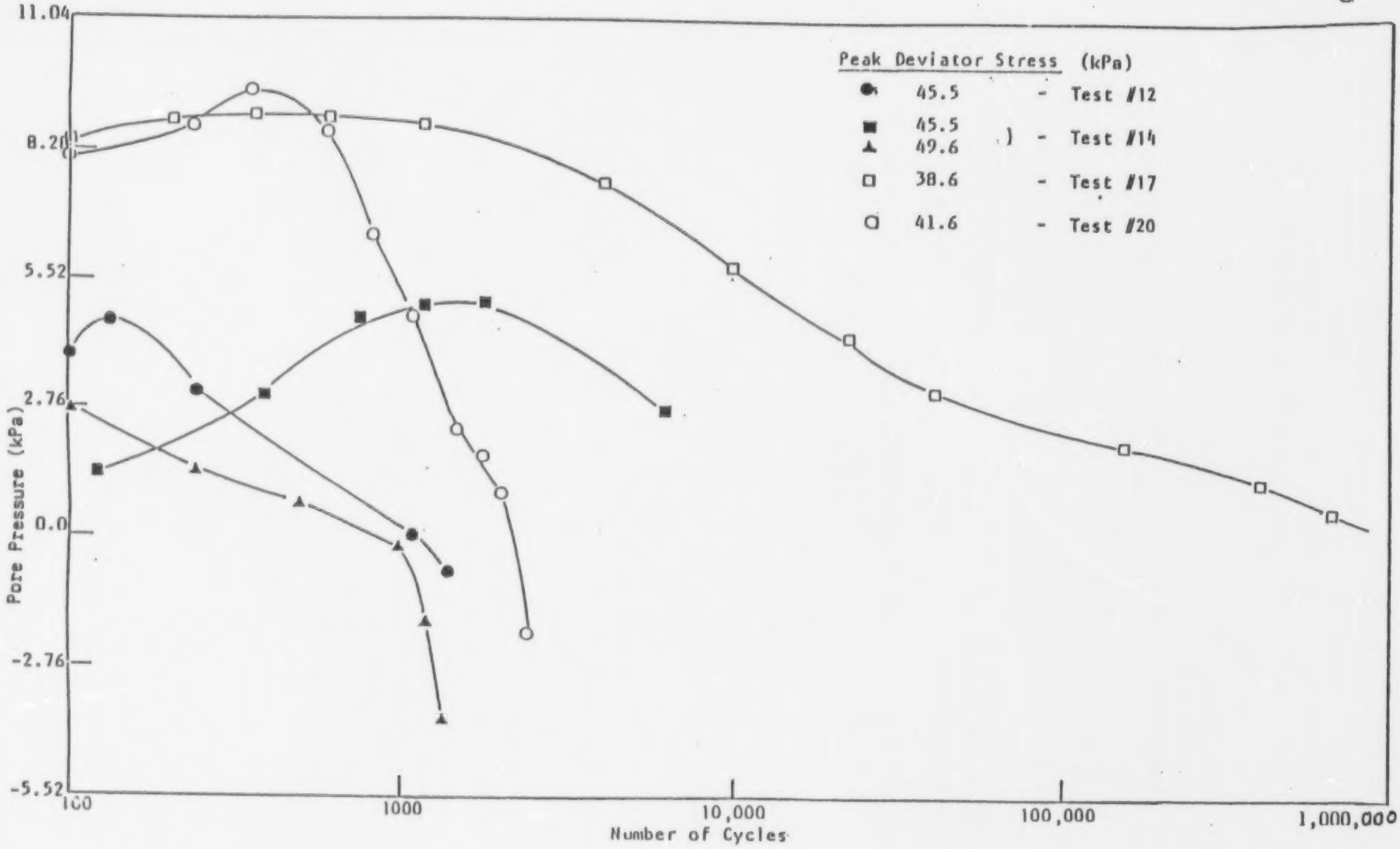


Fig. 36 VARIATION IN PORE PRESSURE WITH NUMBER OF LOAD CYCLES FOR SPECIMENS FROM COMPOSITE SAMPLE "B". (σ_3 = 13.8 kPa)

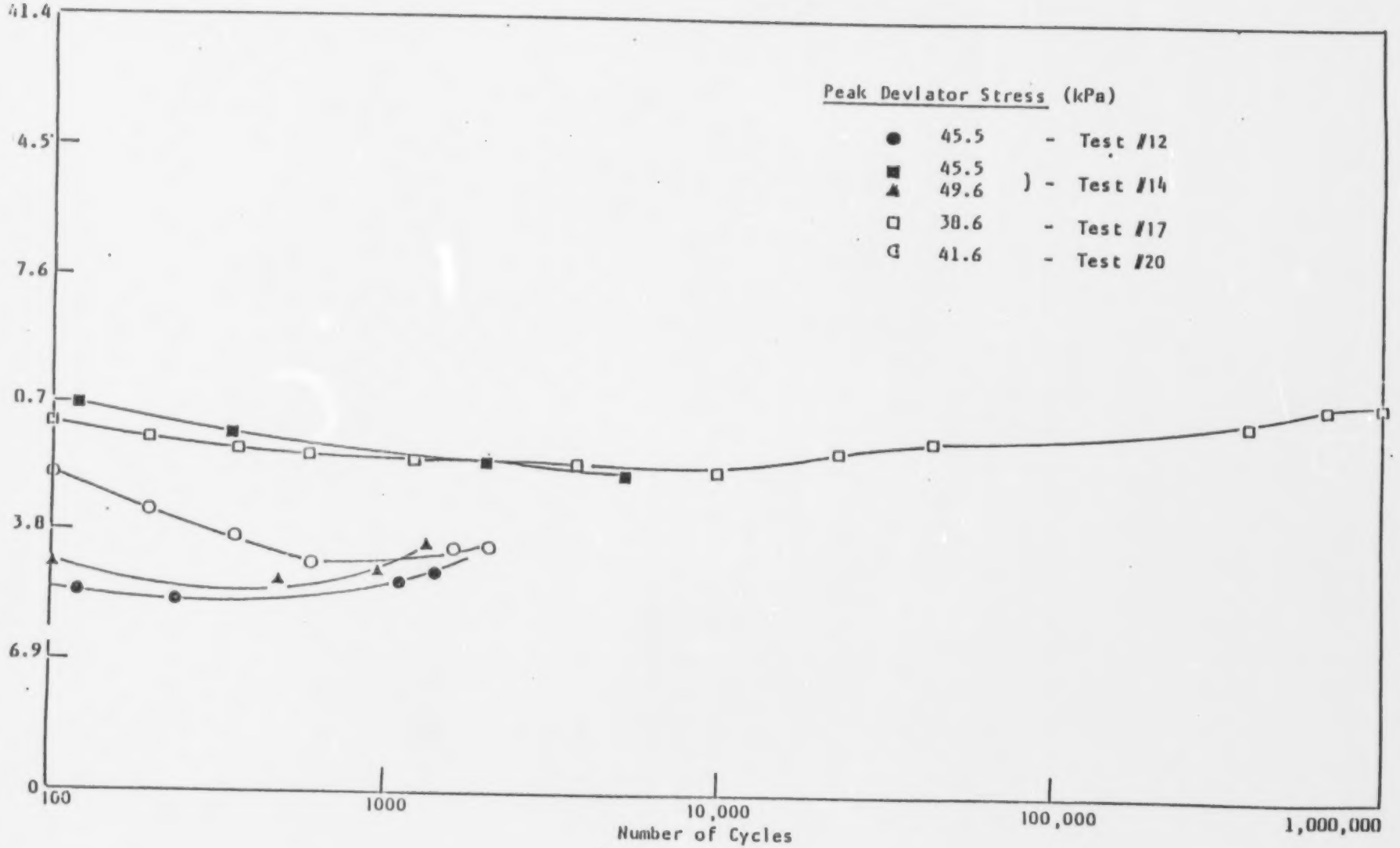


Fig. 37 VARIATION IN DYNAMIC MODULUS WITH NUMBER OF LOAD CYLES FOR SPECIMENS FROM COMPOSITE SAMPLE "B".

Soil Type	Test Condition	Consolidation Pressure (kPa)	Back Pressure (kPa)	Peak Cyclic Deviator Stress (kPa)	No. of Cycles	Cumulative Strain (%)	Range of Elastic Strain (%)	Range of Dynamic Modulus (MPa)	Range of Pore Pressure (kPa)	Initial Water Content (%)	Final Water Content (%)	Dry Density (Mg/m^3)	Remarks
A	undrained	13.8	275.8	34.5	500000	0.83	0.24-0.13	14.9-26.9	10.1-1.9	25.7	29.1	1.56	not failed
A	"	13.8	275.8	37.9	4500	7.90	0.38-.43-.37	9.8-10.3	8.1-11.4-2.6	26.0	29.1	1.56	failed
A	"	13.8	275.8	41.4	1300	7.56	0.37-0.55	11.1-8.0	N.A.	25.1	N.A.	1.55	failed
A	"	27.6	275.8	45.5	400000	0.55	0.8-.04	8.1-16.5	15.9-21.0-2.2	25.4	29.0	1.56	not failed
A	"	27.6	275.8	49.6	4000	6.30	0.26-0.34	19.8-13.3	N.A.	25.0	N.A.	1.56	failed
A	"	27.6	275.8	51.7	2000	8.30	0.38-0.56	13.4-9.1	16.9-6.9	25.0	N.A.	1.58	failed
A	"	27.6	275.8	58.7	1300	5.59	0.45-0.62	14.5-9.6	N.A.	25.1	28.9	1.57	failed
A	"	41.4	275.8	48.3	400000	0.54	0.22-.27-.16	22.5-30.5	11.6-19.8	25.0	27.3	1.60	not failed
A	"	41.4	275.8	51.7	350000	1.07	0.10-.07	50.0-75.0	8.1-3.4-9.5	25.1	29.2	1.55	not failed
A	"	41.4	275.8	55.2	500	8.43	.42-.46	13.0-12.0	16.3-1.2	25.3	28.1	1.55	failed

TABLE 7 Dynamic Triaxial Test Parameters for Specimens Prepared from Composite Sample "A" (Truck Pulse)

Fig. 38 Relationship Between Cumulative Axial Strain and Number of Load Cycles for Soil "A" (σ_3 = 13.8 kPa - Truck Pulse)

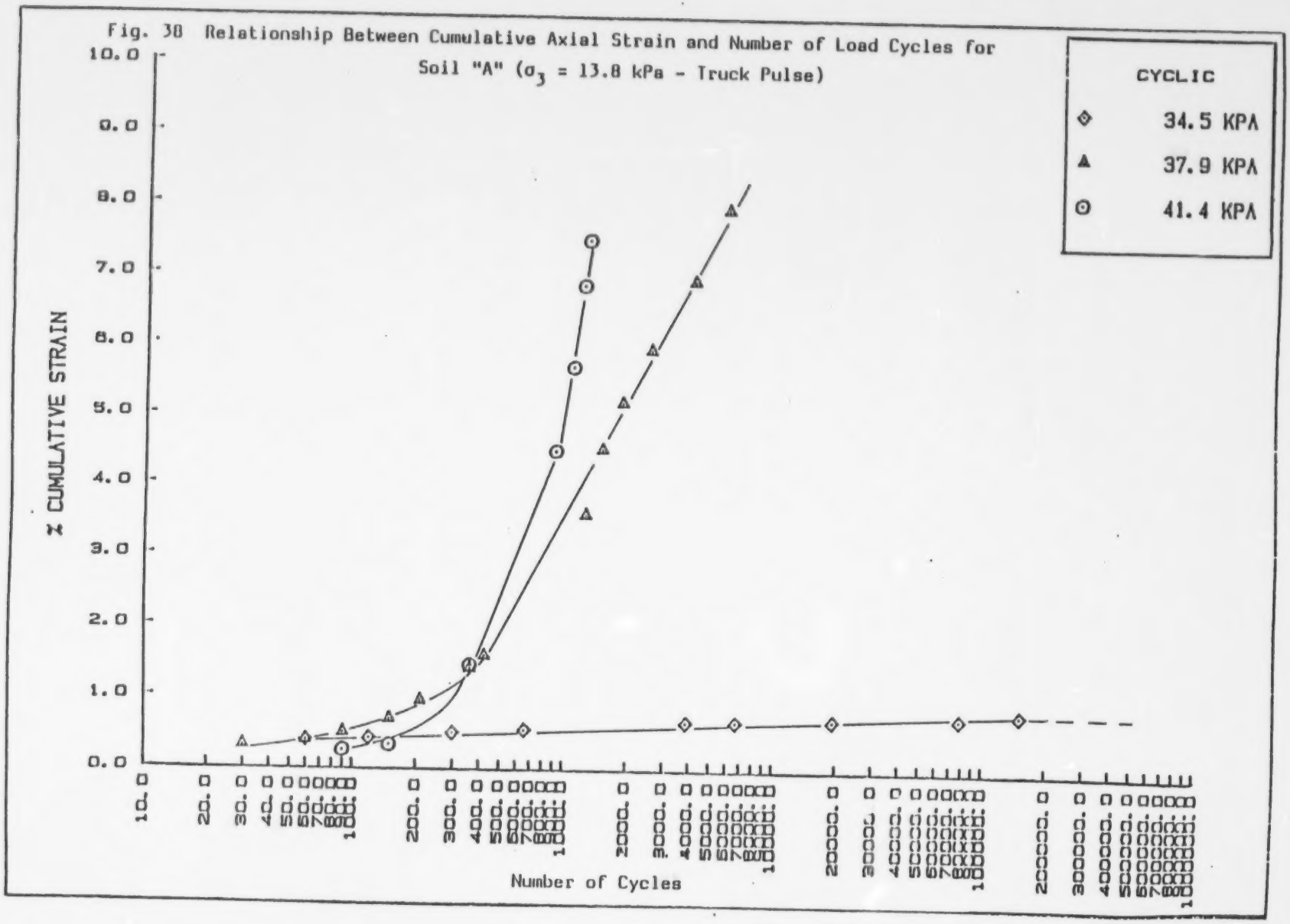


Fig. 39 Elastic Strain versus Number of Load Cycles for Soil "A" (σ_3 = 13.8 kPa Truck Pulse)

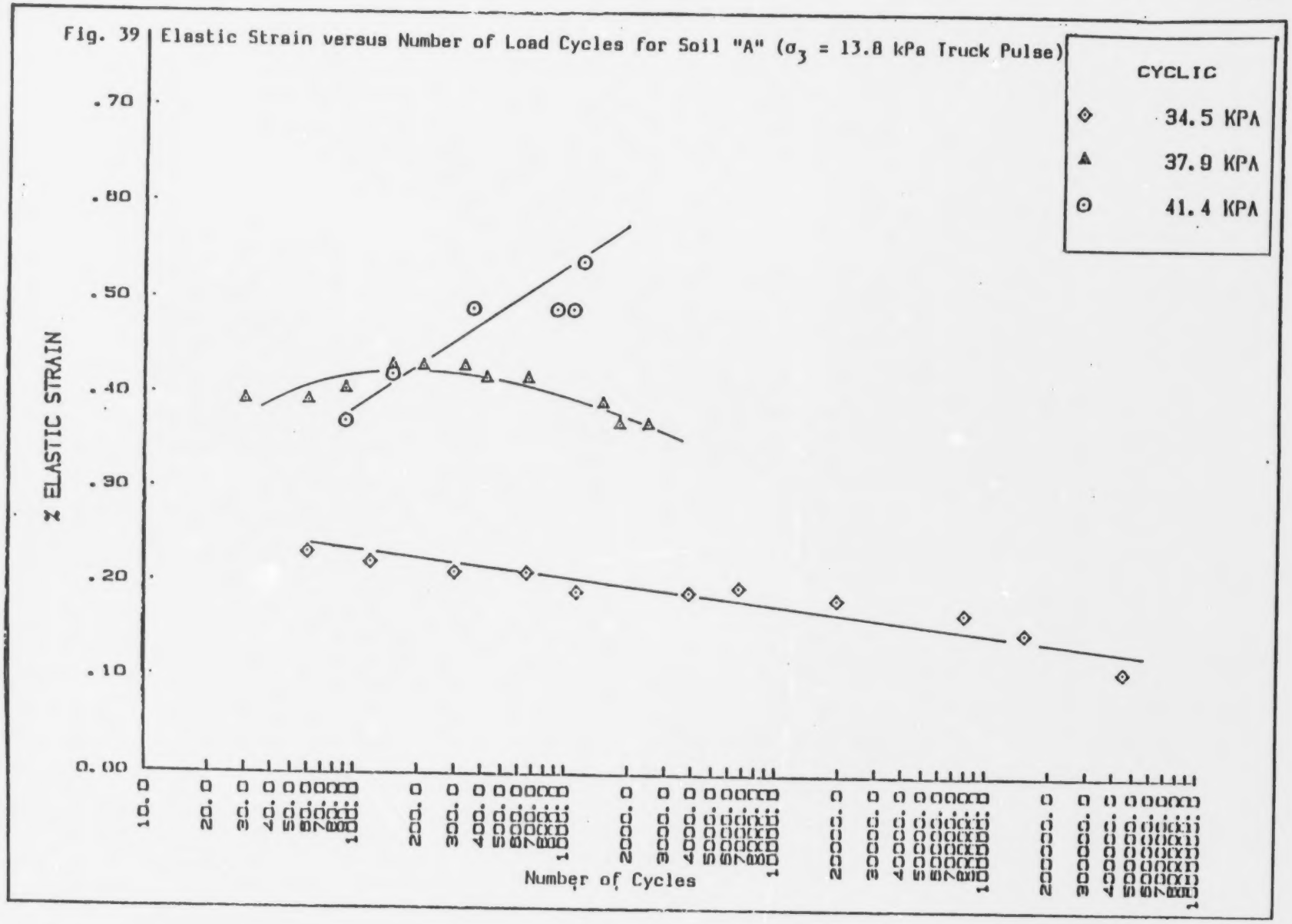


Fig. 40 Pore Water Pressure versus Number of Load Cycles for Soil "A" (σ_3 = 13.8 kPa Truck Pulse)

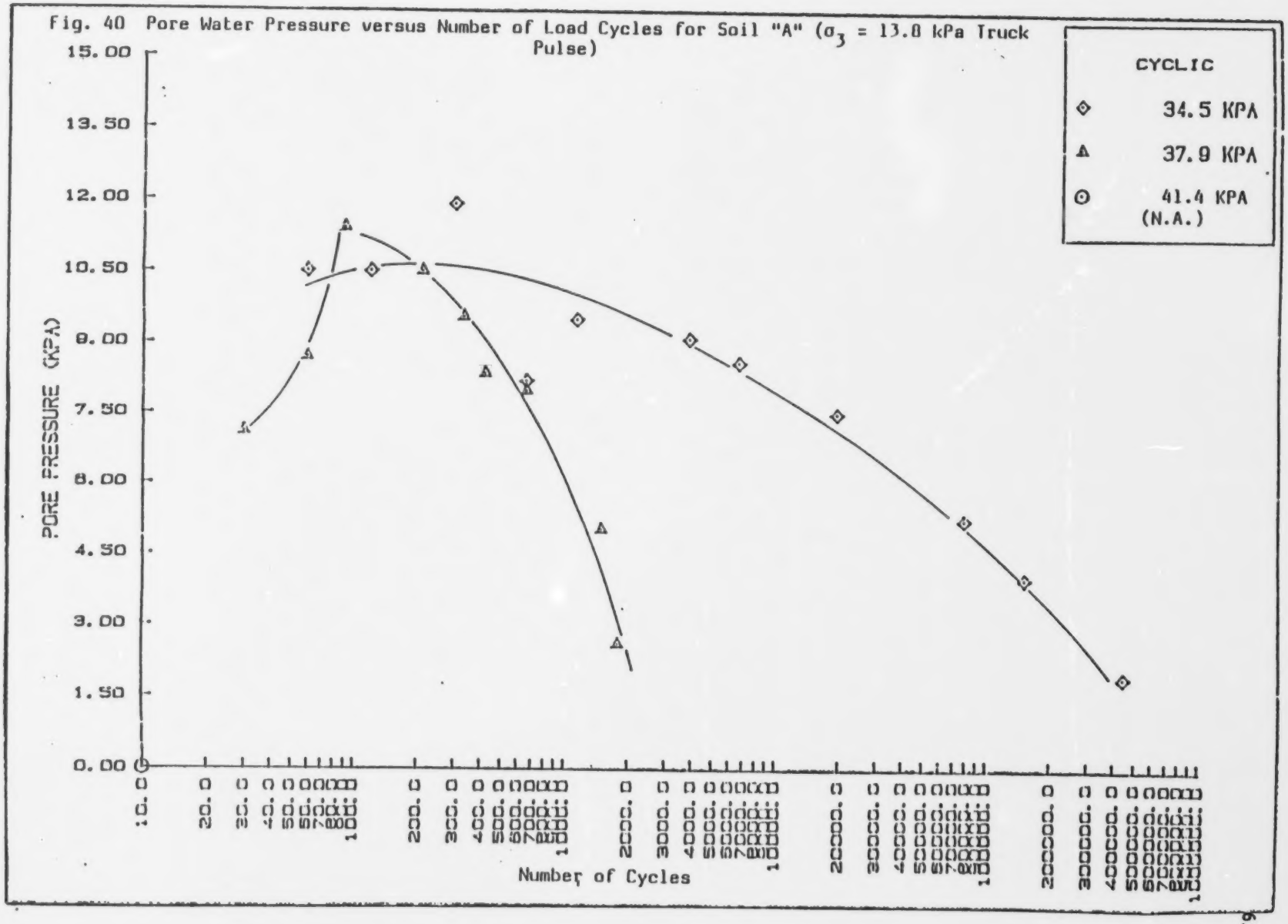


Fig. 41 Dynamic Modulus versus Number of Load Cycles for Soil "A" (σ_3 = 13.8 kPa Truck Pulse)

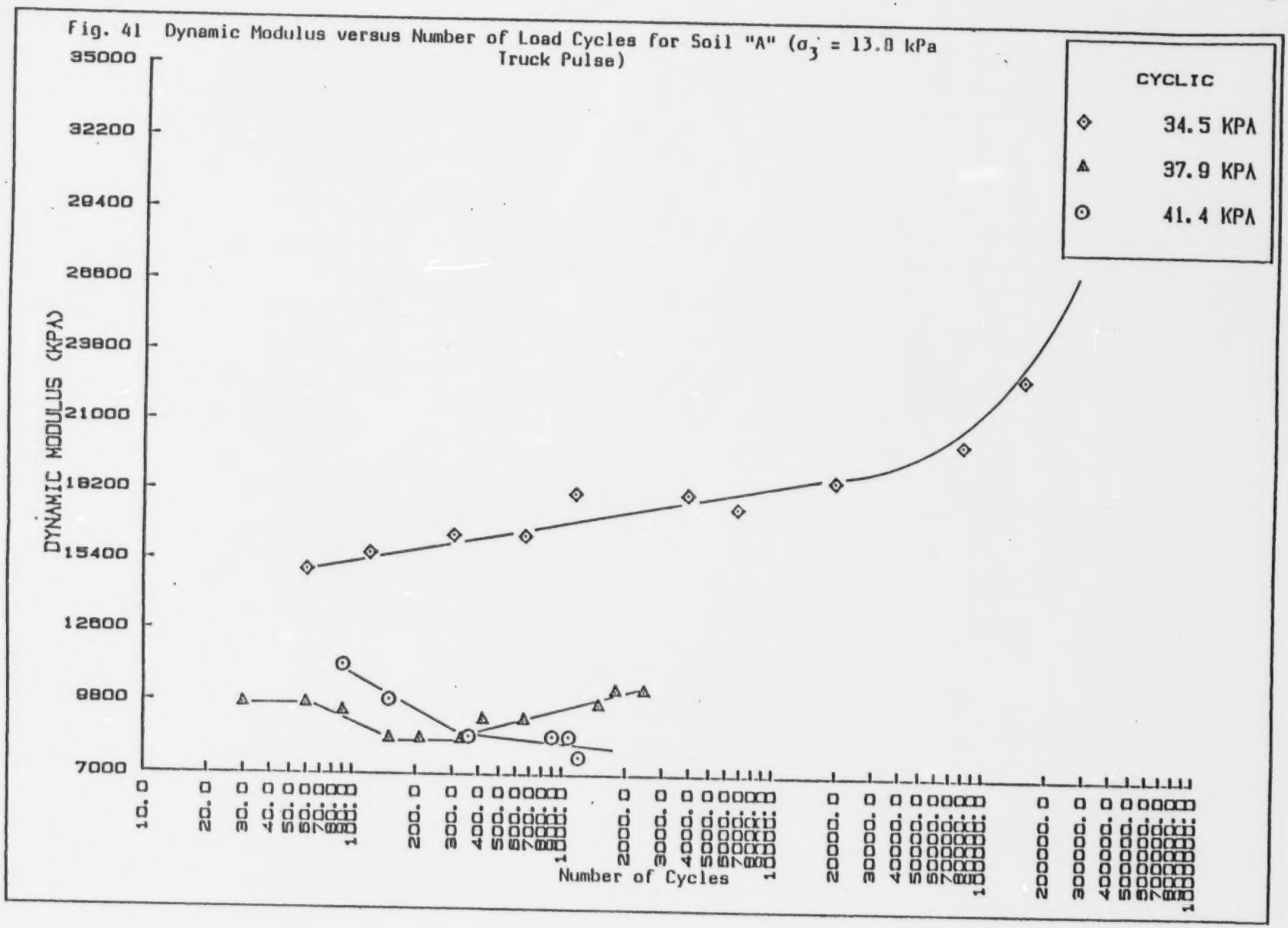


Fig. 42 Relationship Between Cumulative Axial Strain and Number of Load Cycles for Soil "A" (σ_3 = 27.6 kPa - Truck Pulse)

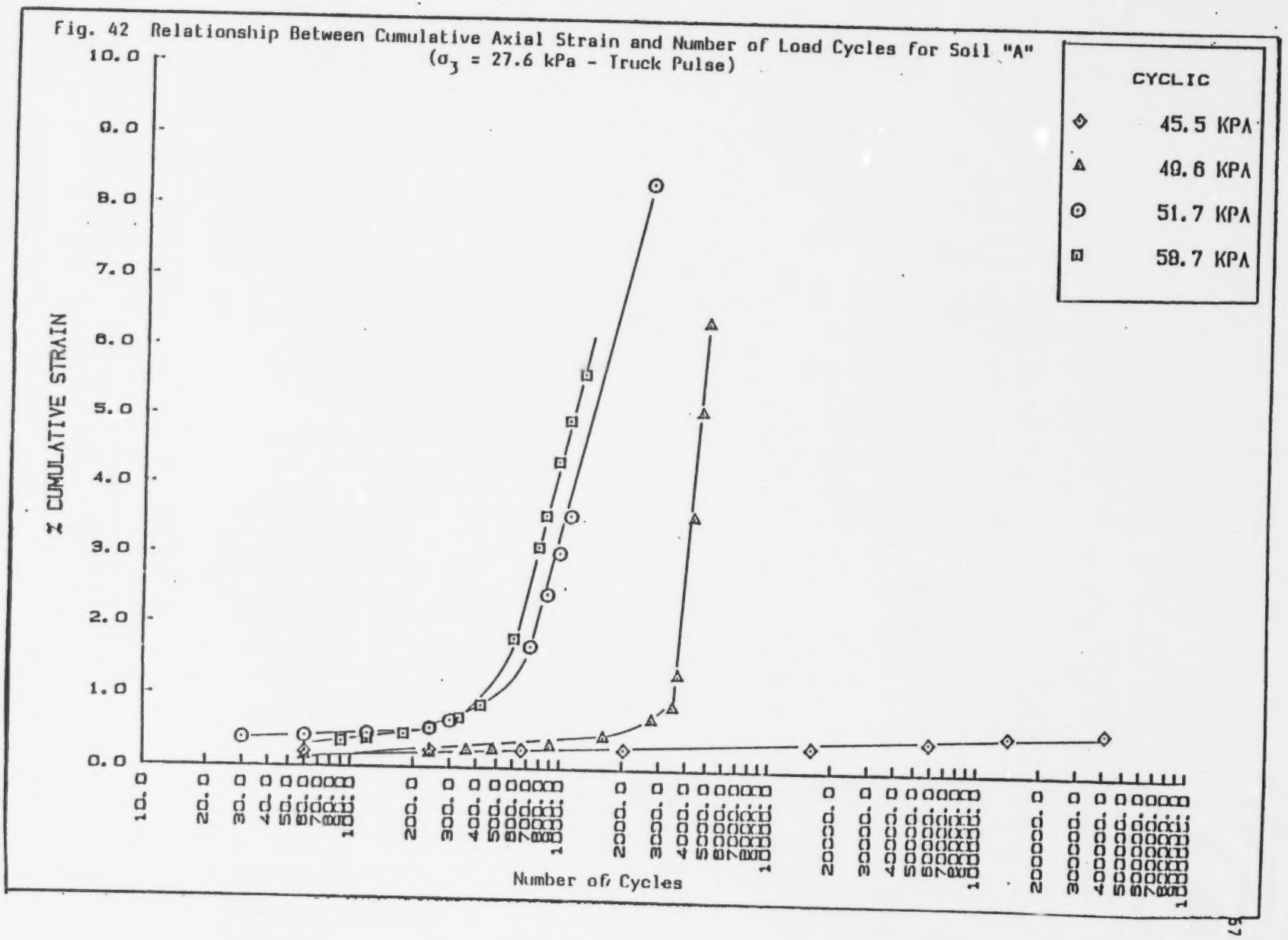


Fig. 43 Elastic Strain versus Number of Load Cycles for Soil "A" (σ_3 = 27.6 kPa - Truck Pulse)

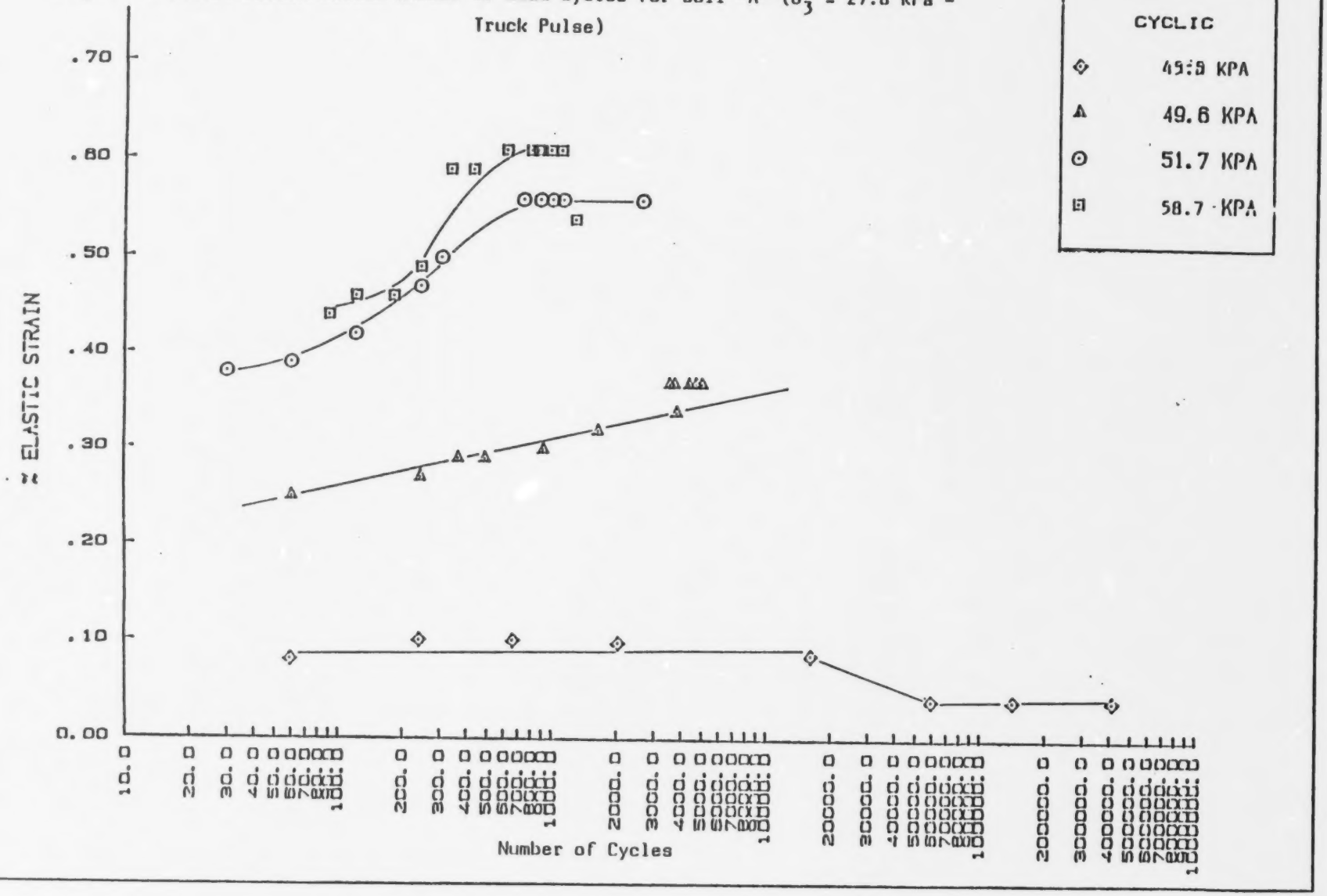


Fig. 44 Pore Water Pressure versus Number of Load Cycles for Soil "A"
(σ_3 = 27.6 kPa - Truck Pulse)

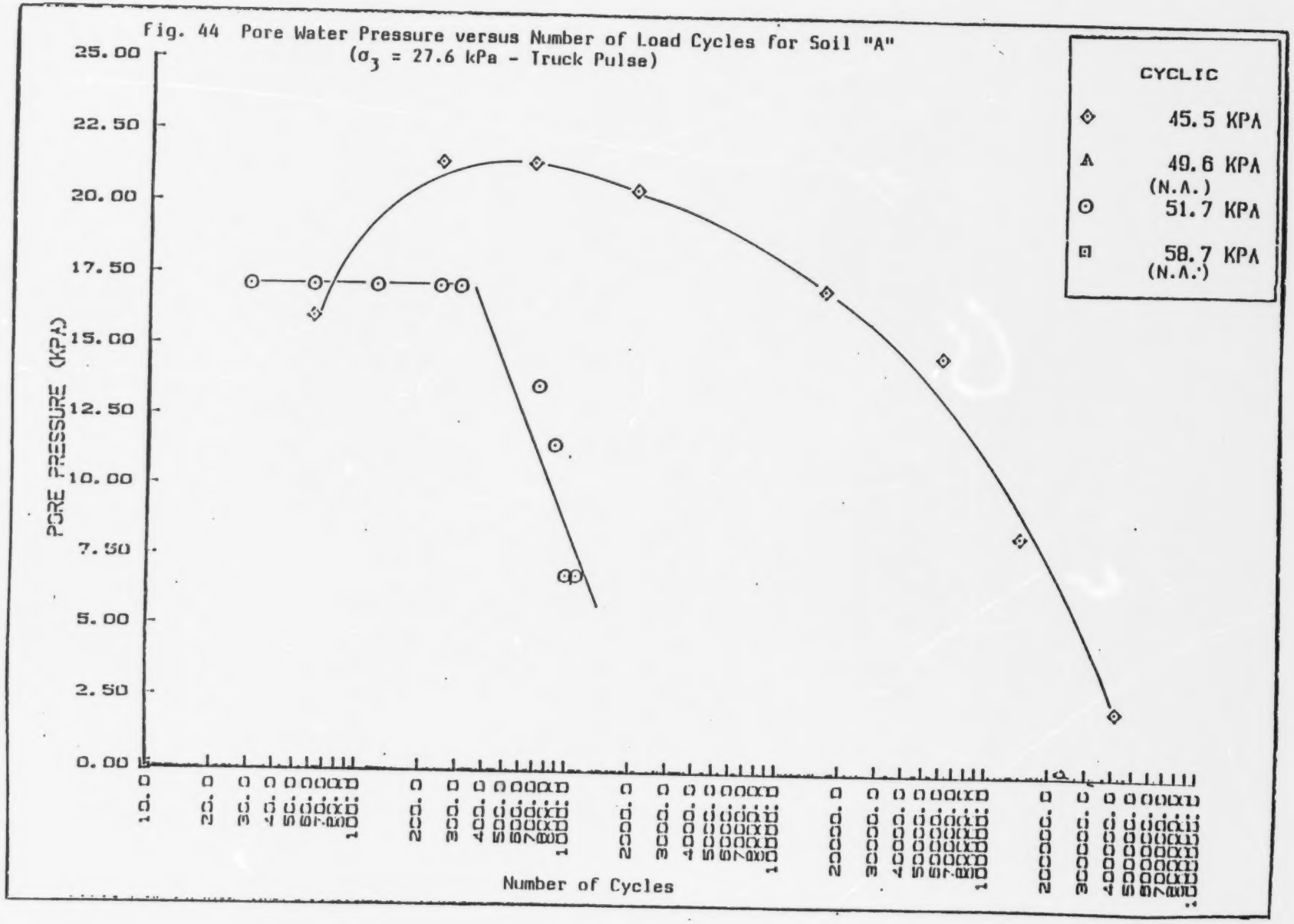


Fig. 45 Dynamic Modulus versus Number of Load Cycles for Soil "A"
(σ_3 = 27.6 kPa - Truck Pulse)

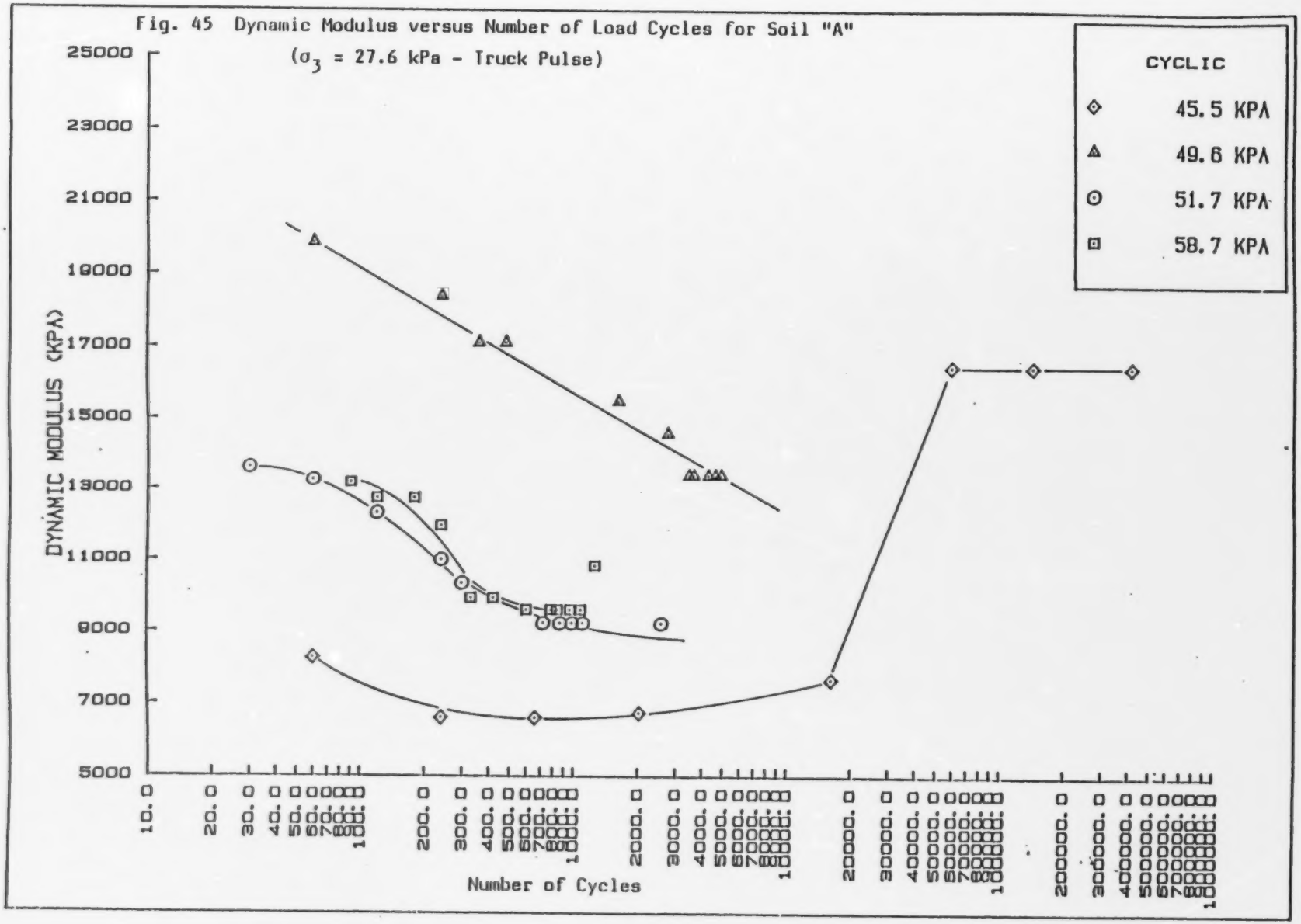


Fig. 46 Relationship Between Cumulative Axial Strain and Number of Load Cycles for Soil "A" (σ_3 = 41.4 kPa - Truck Pulse)

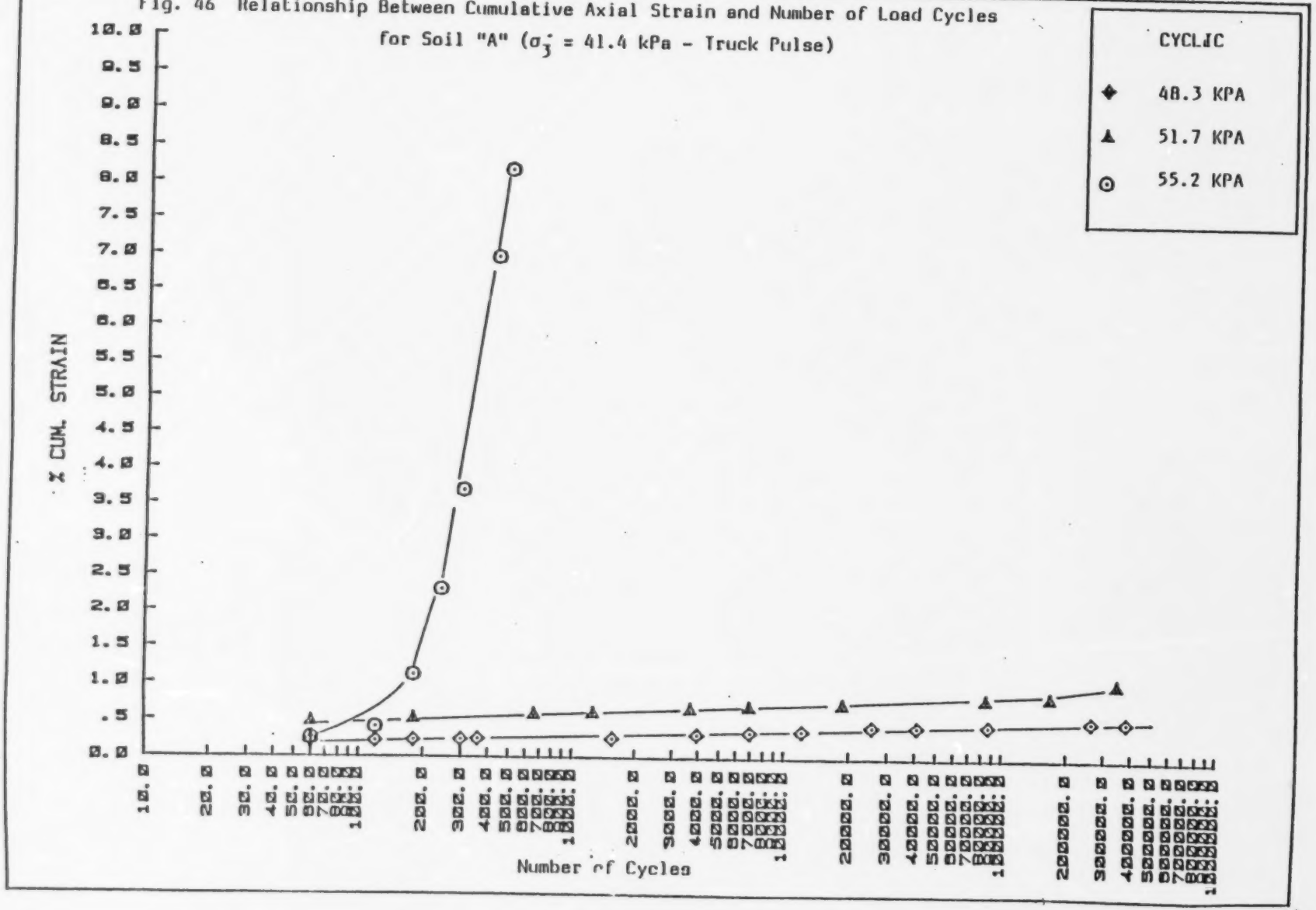


Fig. 47 Elastic Strain versus Number of Load Cycles for Soil "A" (σ_3 = 41.4 kPa - Truck Pulse)

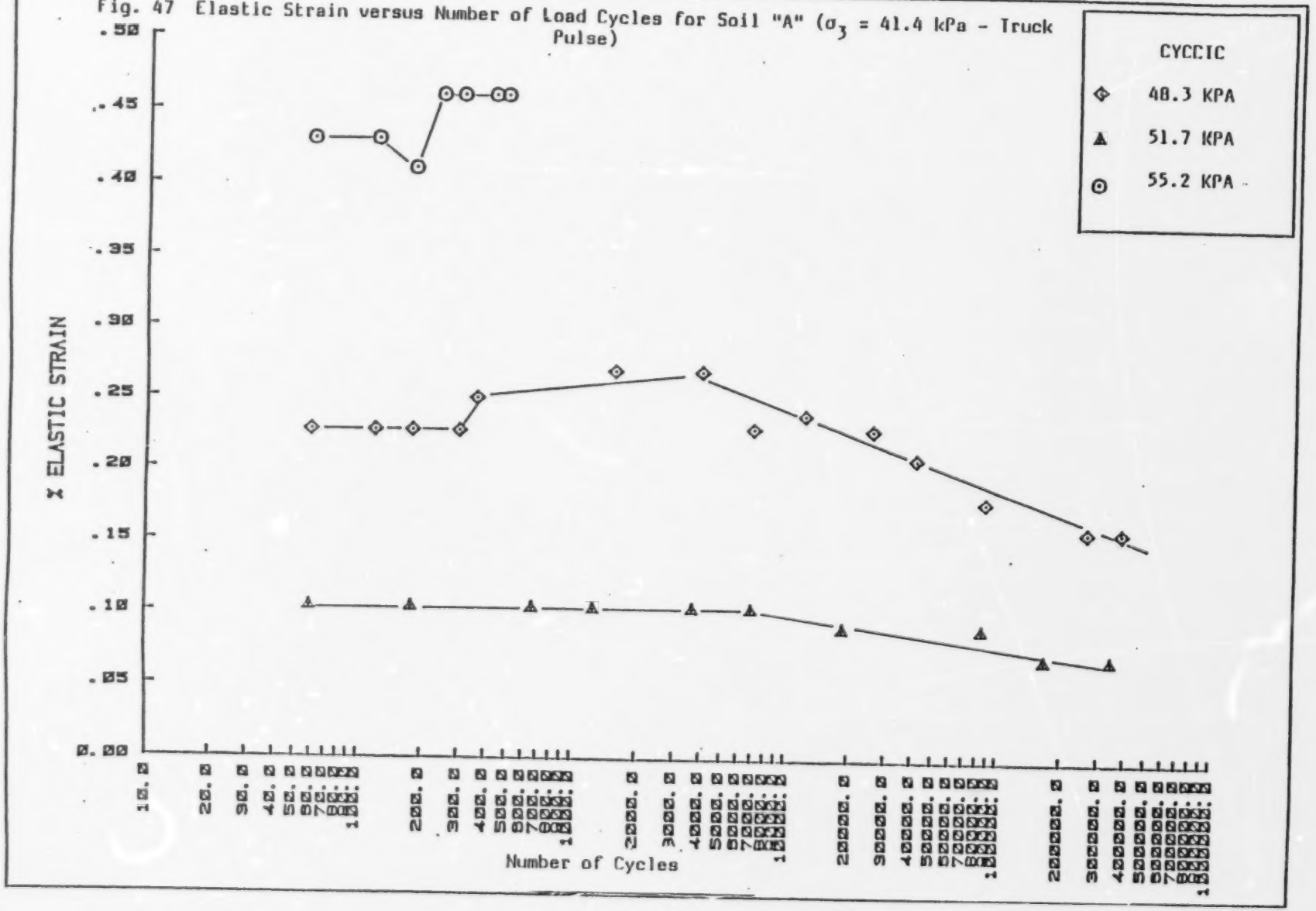


Fig. 48 Pore Water Pressure versus Number of Load Cycles for Soil "A"
(σ_3 = 41.4 kPa - Truck Pulse)

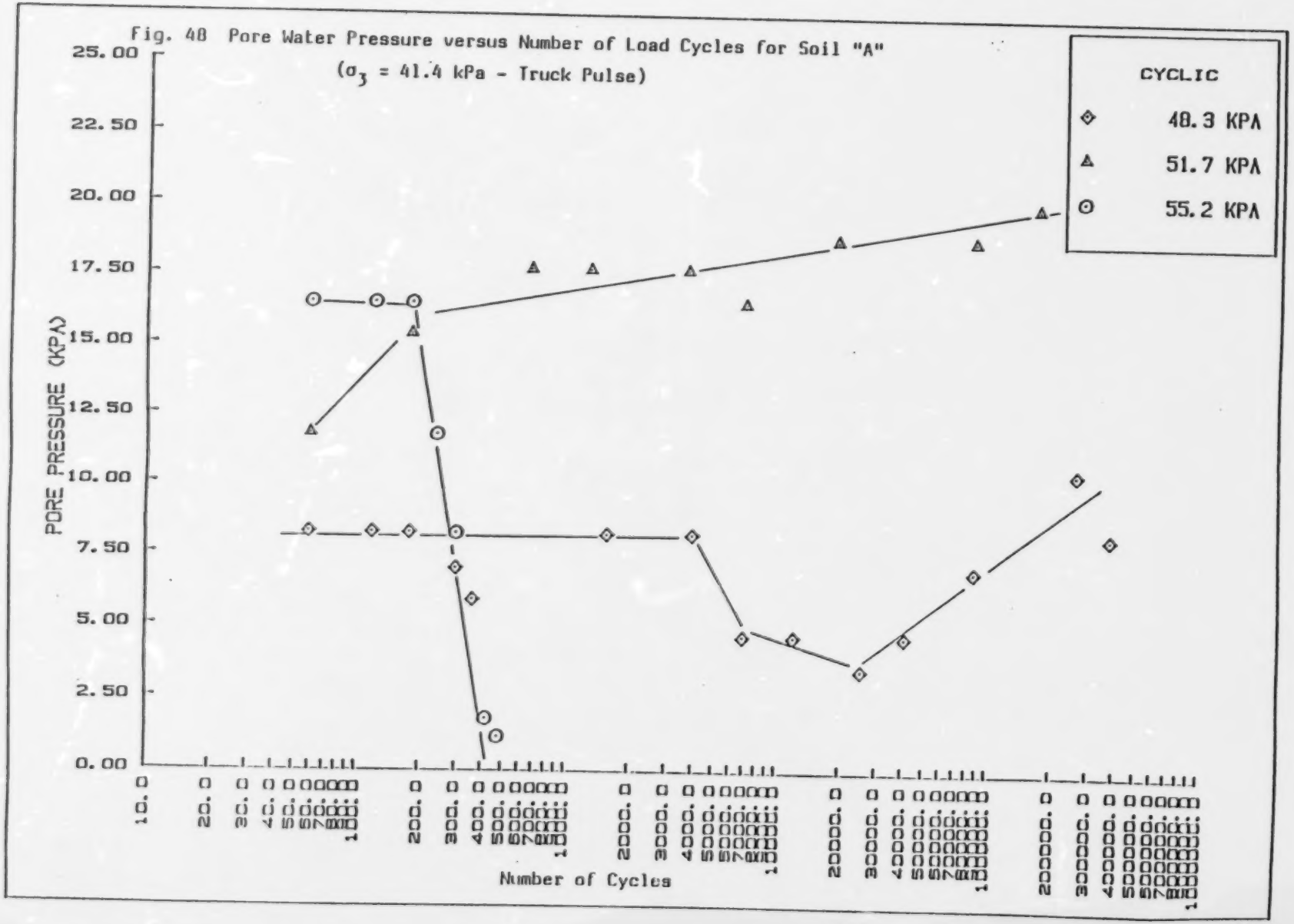
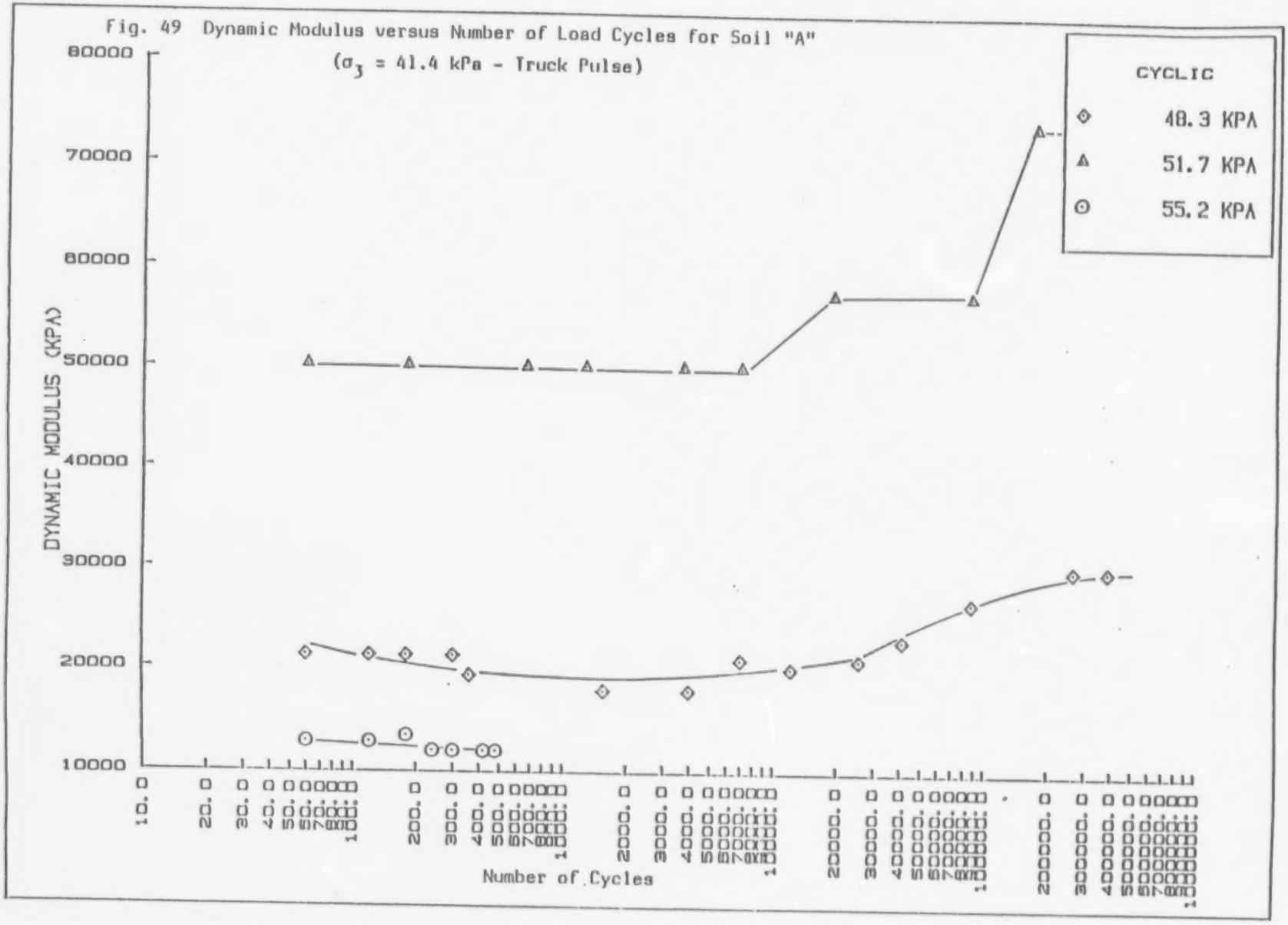


Fig. 49 Dynamic Modulus versus Number of Load Cycles for Soil "A"
(σ_3 = 41.4 kPa - Truck Pulse)



Soil Type	Test Condition	Consolidation Pressure (kPa)	Back Pressure (kPa)	Peak Cyclic Deviator Stress (kPa)	No. of Cycles	Cumulative Strain (%)	Range of Elastic Strain (%)	Range of Dynamic Modulus (MPa)	Range of Pore Pressure (kPa)	Initial Water Content (%)	Final Water Content (%)	Dry Density (Mg/m^3)	Remarks
B	undrained	13.8	275.8	37.2	450000	1.66	.42-.24	8.9-15.5	6.7-0	26.0	33.0	1.49	not failed
B	"	13.8	275.8	38.6	7000	7.55	.30-.46-.41	12.8-8.5-11.1	9.4-5.2	26.2	33.0	1.54	failed
B	"	13.8	275.8	40.1	3200	8.28	.23-.25	17.4-15.9	5.1-10.0-(-.4)	26.6	27.5	1.52	failed
B	"	13.8	275.8	42.5	2800	12.2	.27-.42	15.7-10.2	4.1-	27.0	35.3	1.46	failed
B	"	27.6	275.8	32.7	2500	5.06	.17-.11	19.2-29.6	13.1-3.5	26.5	35.3	1.49	failed
B	"	27.6	275.8	40.9	400	6.83	0.23-.53	17.7-9.1	15.9-10.2	26.6	38.1	1.52	failed
B	"	41.4	275.8	48.3	420000	0.33	0.17	29.7-29.8	16.3-15.7	27.0	36.1	1.45	not failed
B	"	41.4	275.8	52.4	6000	6.00	0.21-.30	24.1-17.1	N.A.	26.5	35.0	1.48	failed
B	"	41.4	275.8	53.2	3500	6.44	0.19-.29	20.3-18.8	16.8-27.0-18.0	26.3	33.3	1.49	failed

TABLE 8 Dynamic Triaxial Test Parameters for Specimens Prepared from Composite Sample "B" (Truck Pulse)

Fig. 50 Relationship Between Cumulative Axial Strain and Number of Load Cycles for Soil B (σ_3 = 13.8 kPa - Truck Pulse)

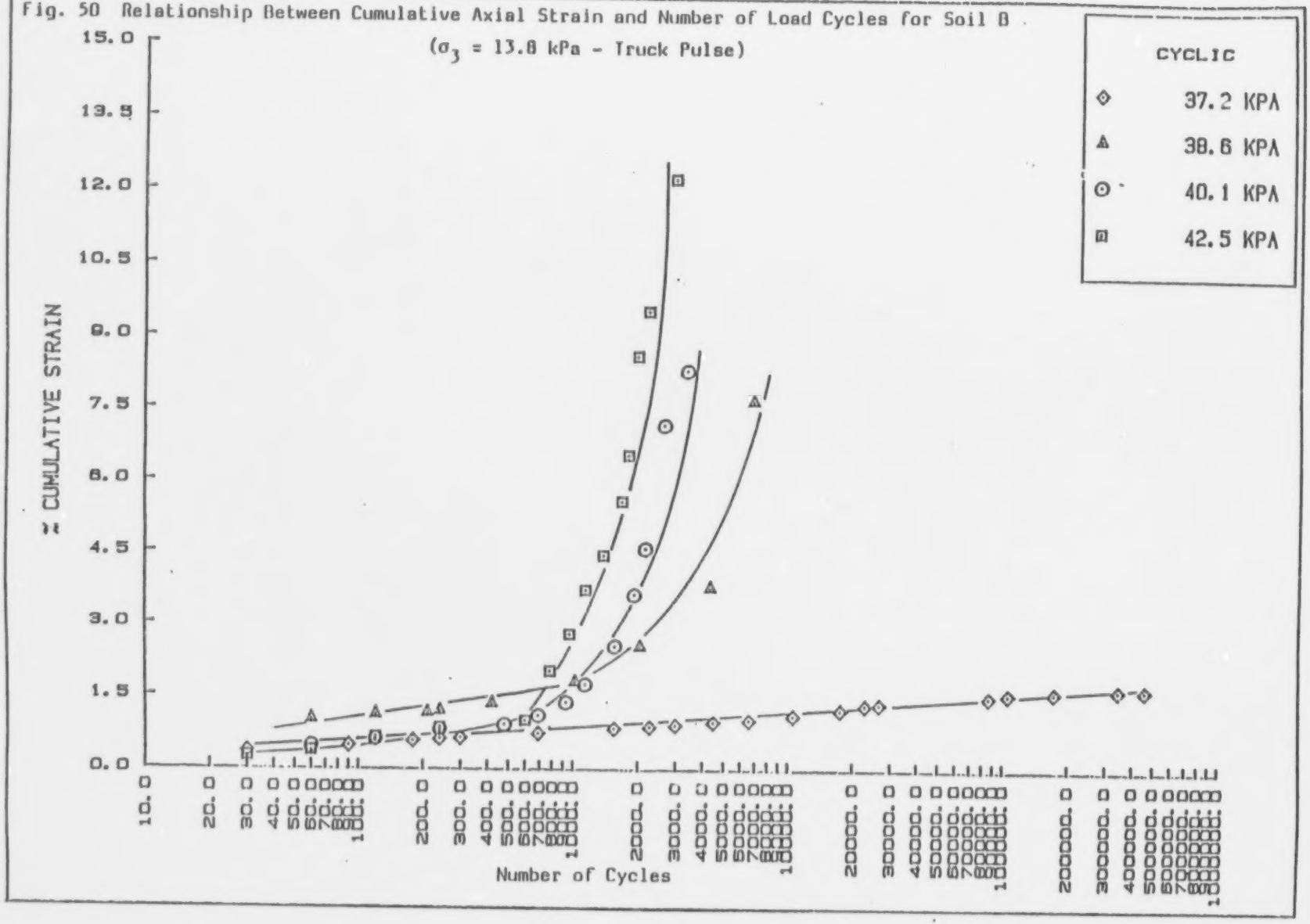


Fig. 51 Elastic Strain versus Number of Load Cycles for Soil B (σ_3 = 13.8 kPa - Truck Pulse)

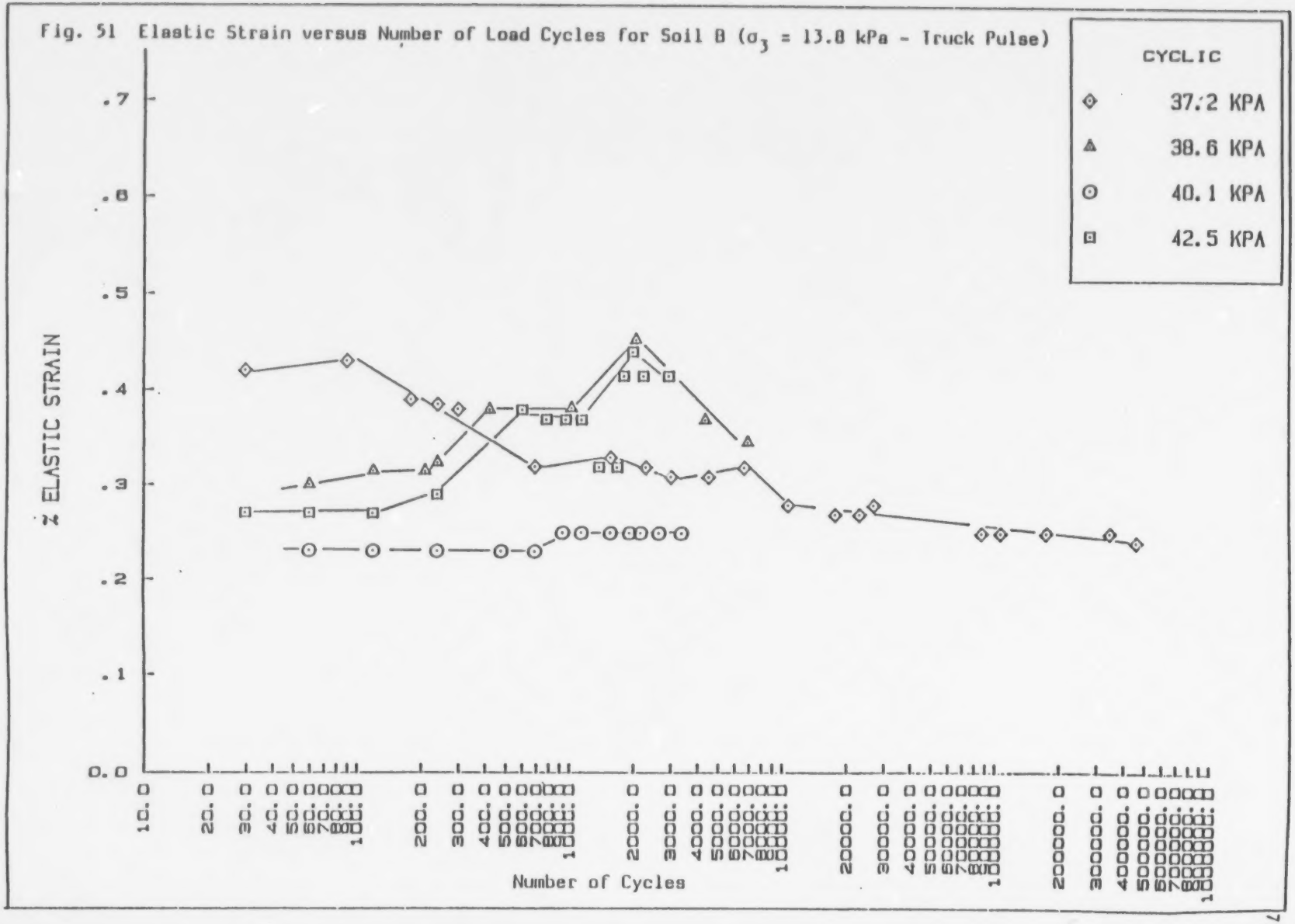


Fig. 52 Pore Pressure versus Number of Load Cycles Soil B (σ_3 = 13.8 kPa - Truck Pulse)

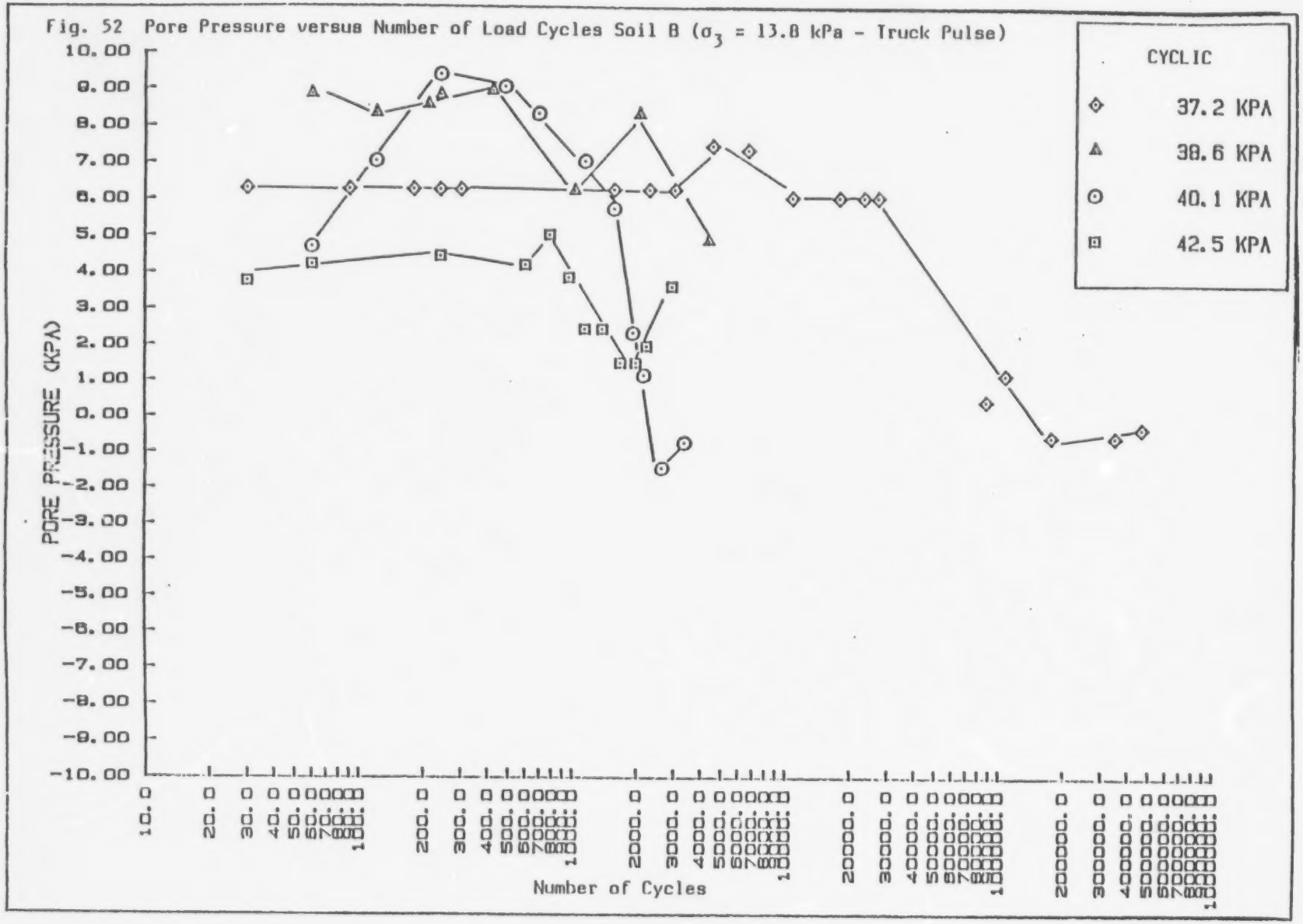


Fig. 53 Dynamic Modulus versus Number of Load Cycles for Soil B (σ_3 = 13.8 kPa - Truck Pulse)

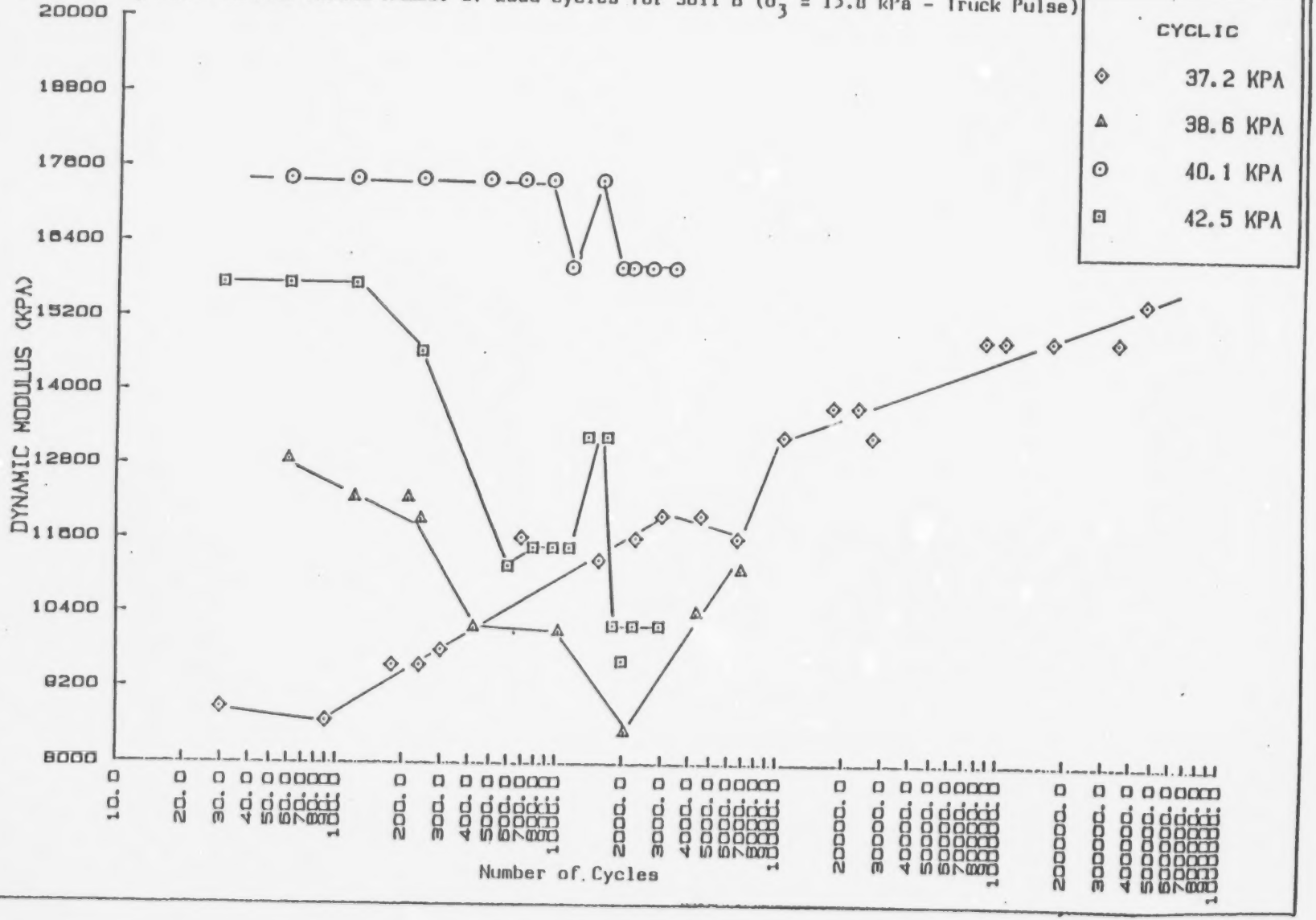


Fig. 54 Relationship Between Cumulative Axial Strain and Number of Load Cycles for Soil B (σ_3 = 27.6 kPa - Truck Pulse)

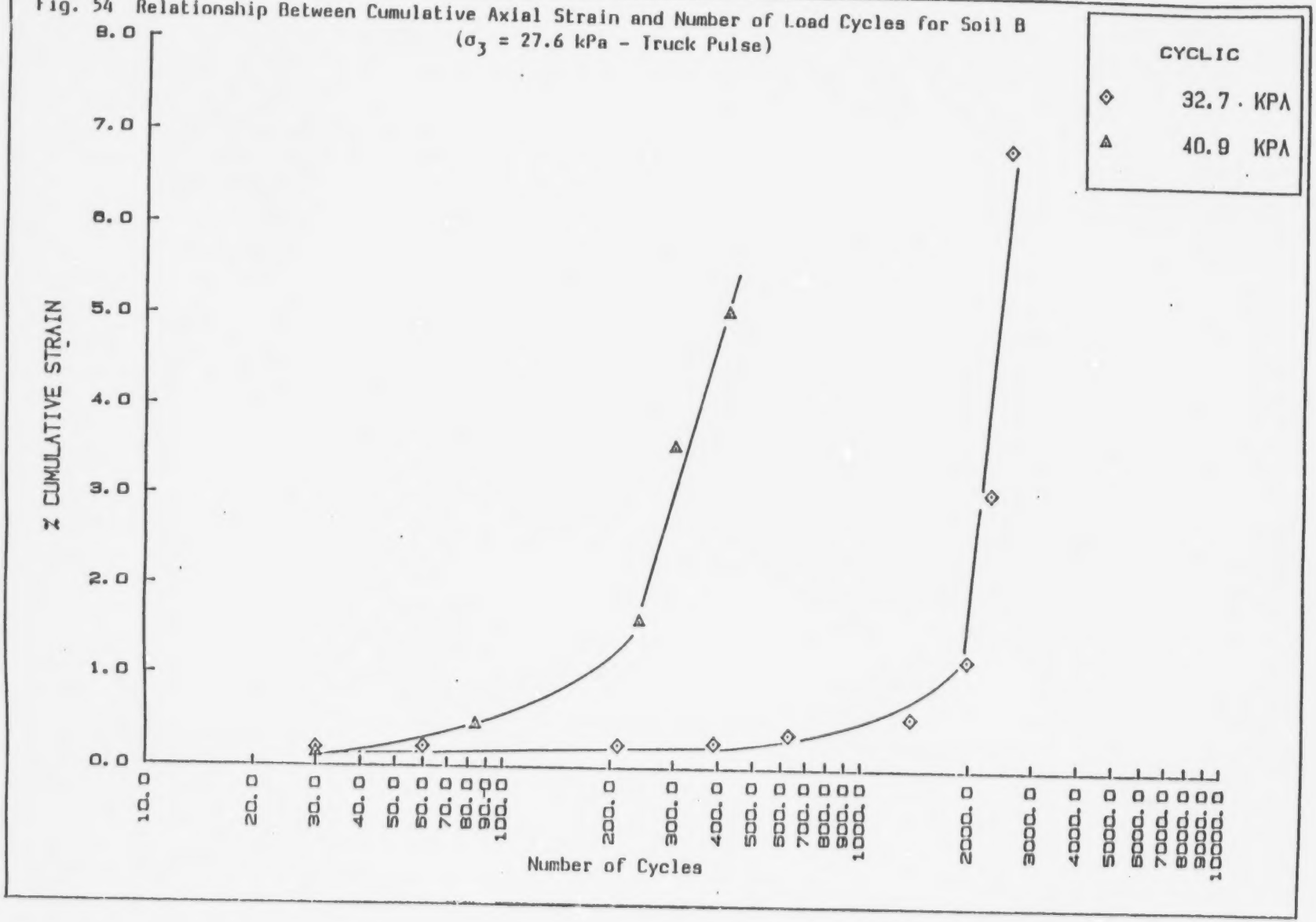


Fig. 55 Elastic Strain versus Number of Load Cycles for Soil B (σ_3 = 27.6 kPa - Truck Pulse)

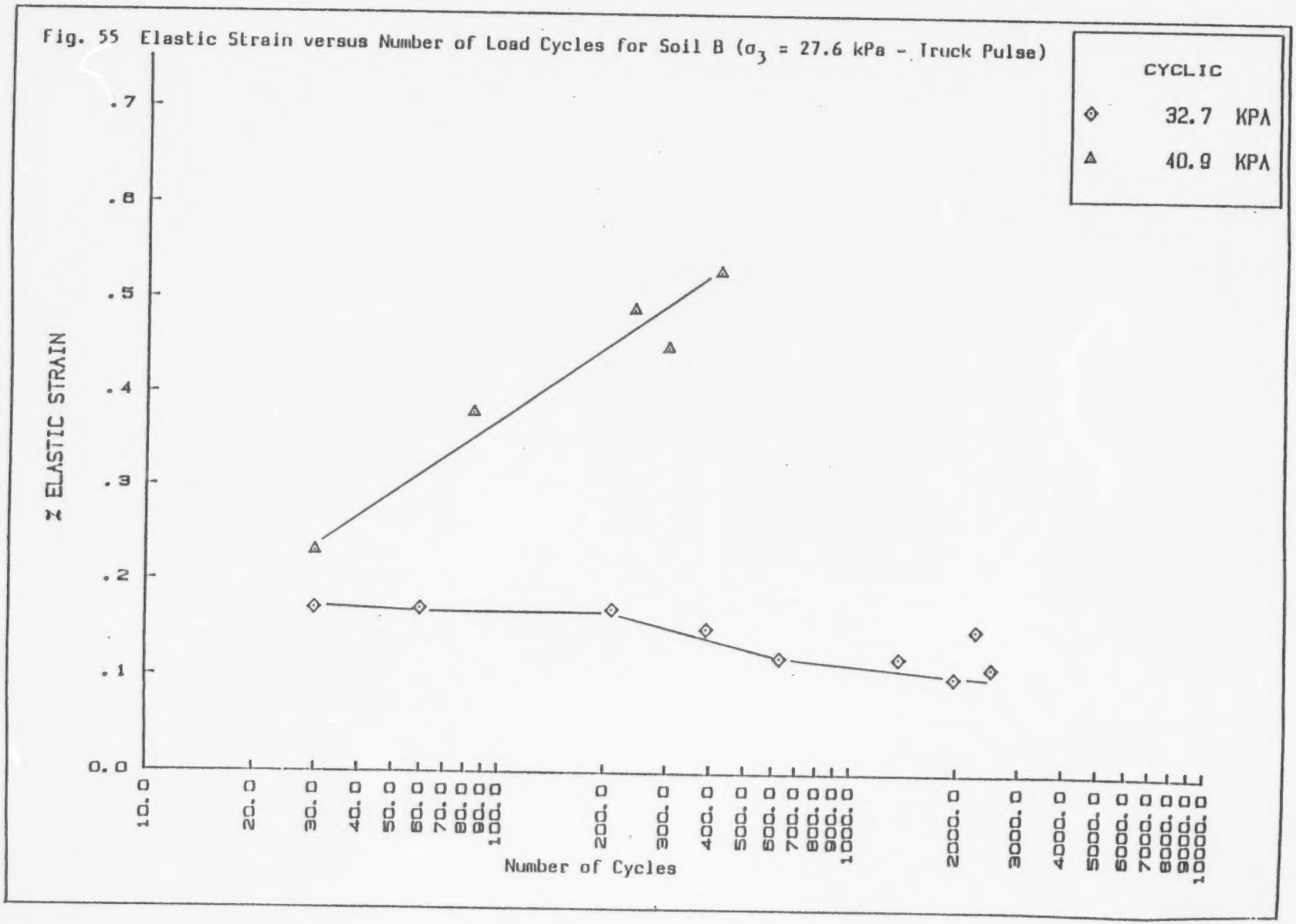
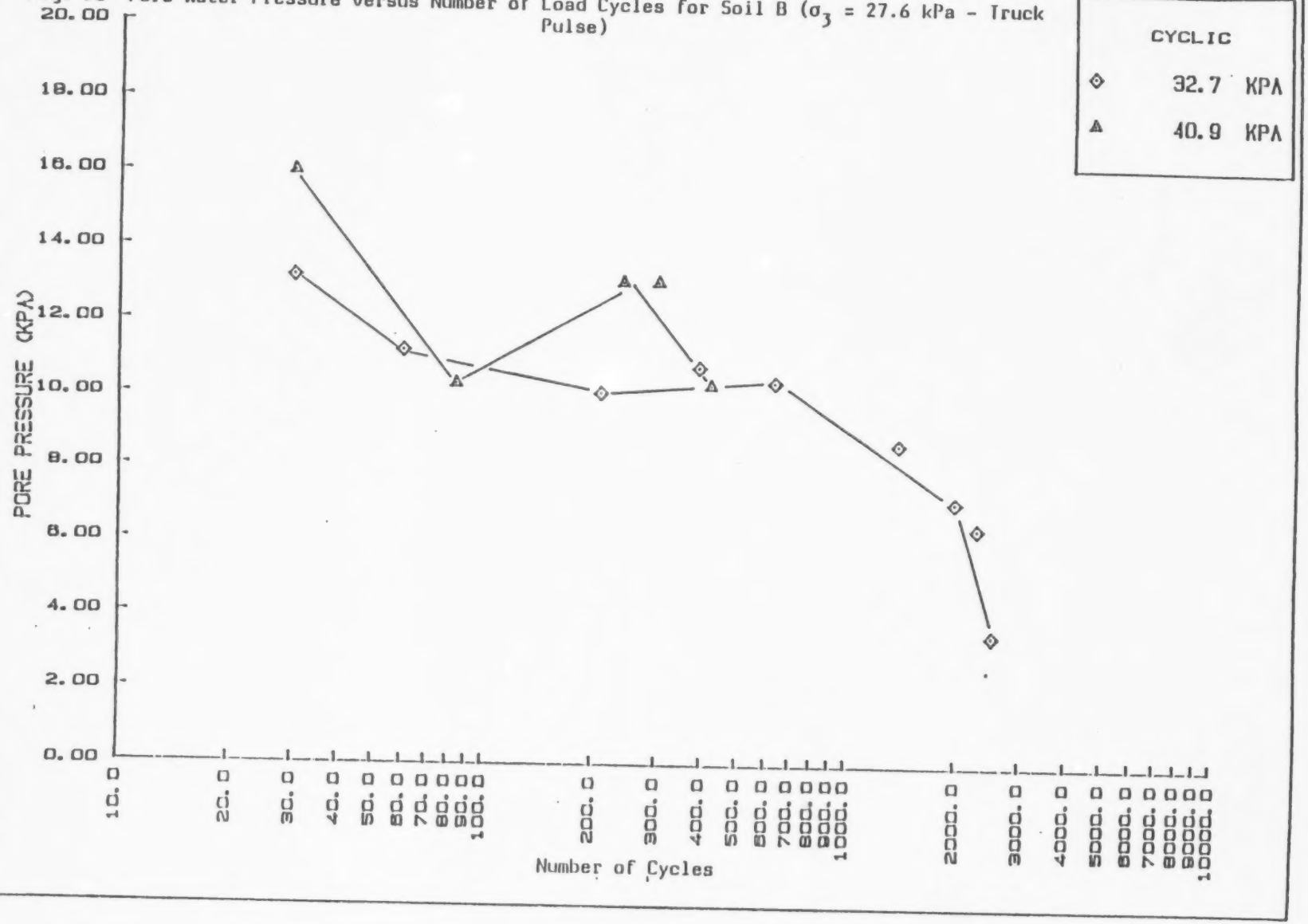


Fig. 56 Pore Water Pressure versus Number of Load Cycles for Soil B (σ_3 = 27.6 kPa - Truck Pulse)



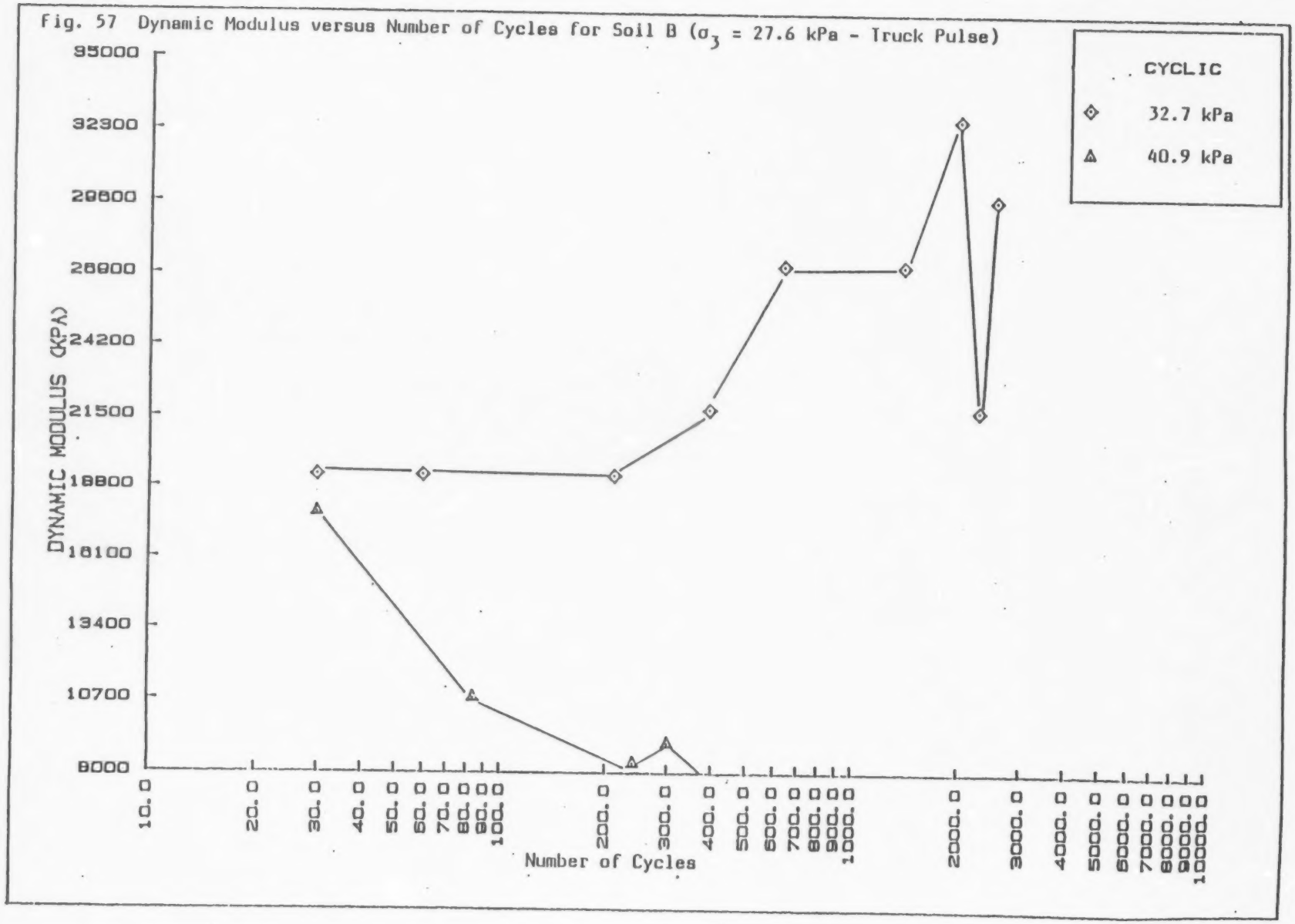


Fig. 57 Dynamic Modulus versus Number of Cycles for Soil B (σ_3 = 27.6 kPa - Truck Pulse)
CYCLIC
32.7 kPa
40.9 kPa
DYNAMIC MODULUS (KPA)
95000
32900
29800
26800
24200
21500
18800
16100
13400
10700
8000
10.0
20.0
30.0
40.0
50.0
60.0
70.0
80.0
90.0
100.
200.0
300.0
400.0
500.0
600.0
700.0
800.0
900.0
1000.
2000.0
3000.0
4000.0
5000.0
6000.0
7000.0
8000.0
9000.0
10000.
Number of Cycles

Fig. 58 Relationship Between Cumulative Axial Strain and Number of Load Cycles for Soil B
(σ_3 = 41.4 kPa - Truck Pulse)

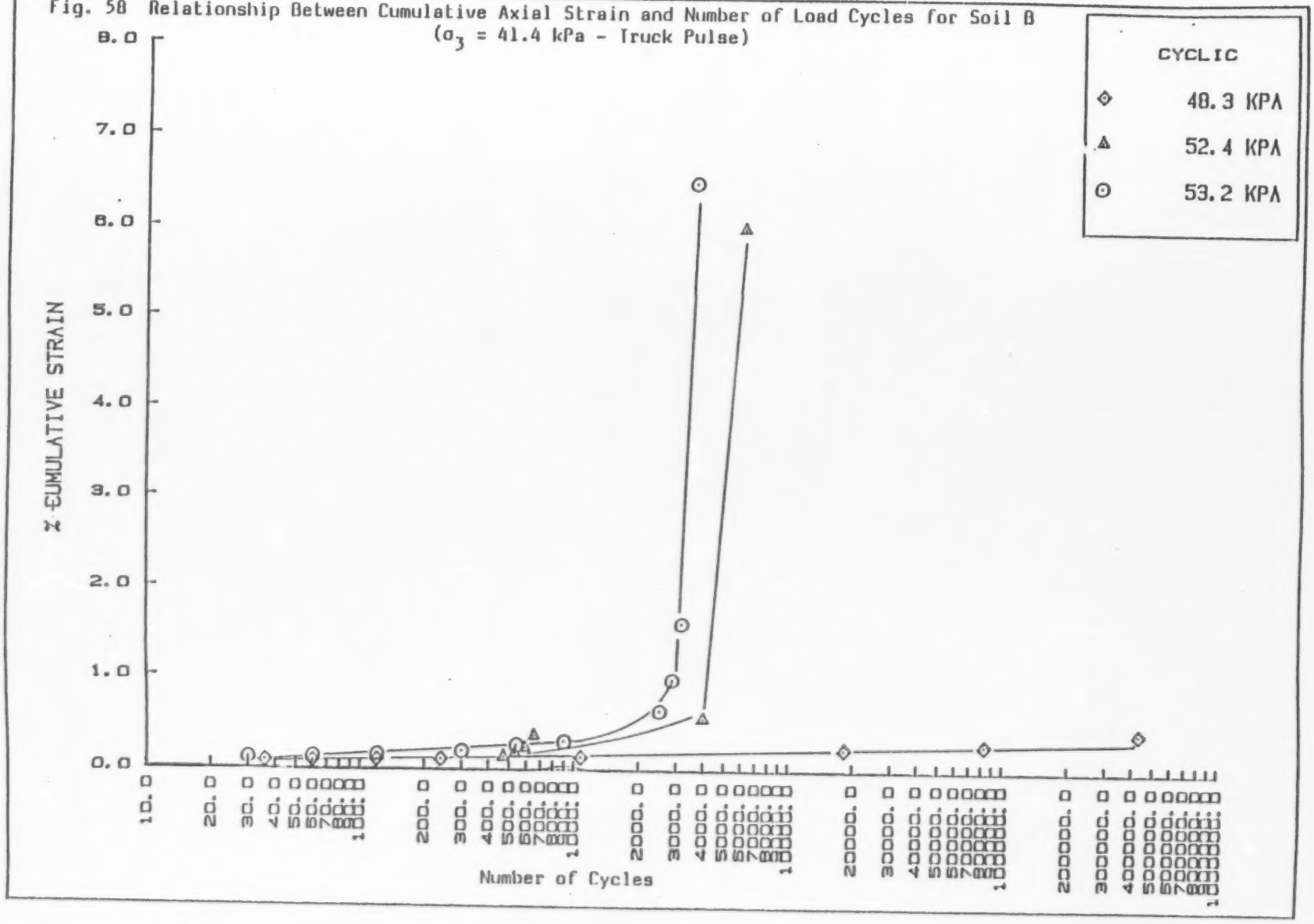


Fig. 59 Elastic Strain versus Number of Load Cycles for Soil B (σ_3 = 41.4 kPa - Truck Pulse)

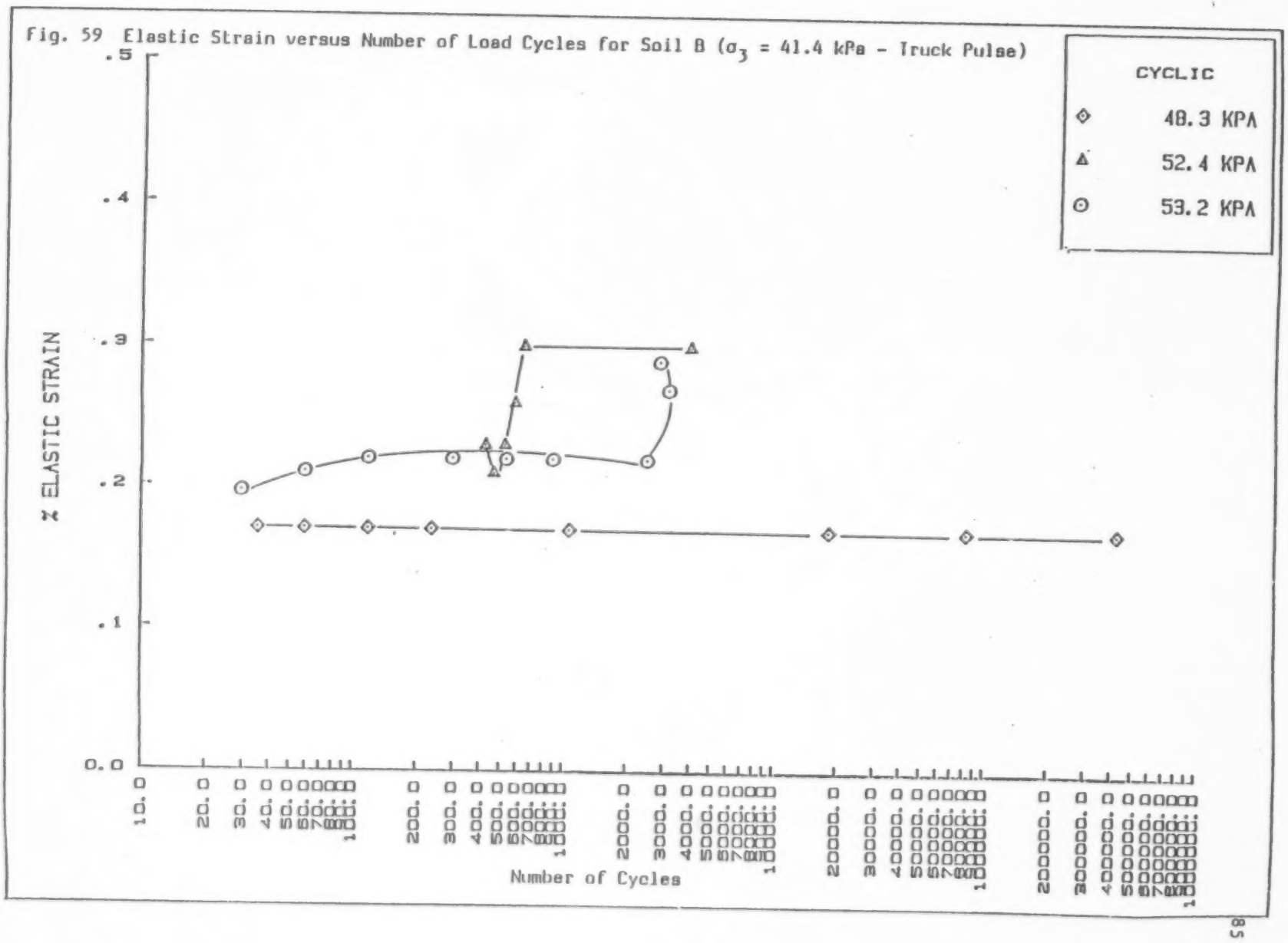
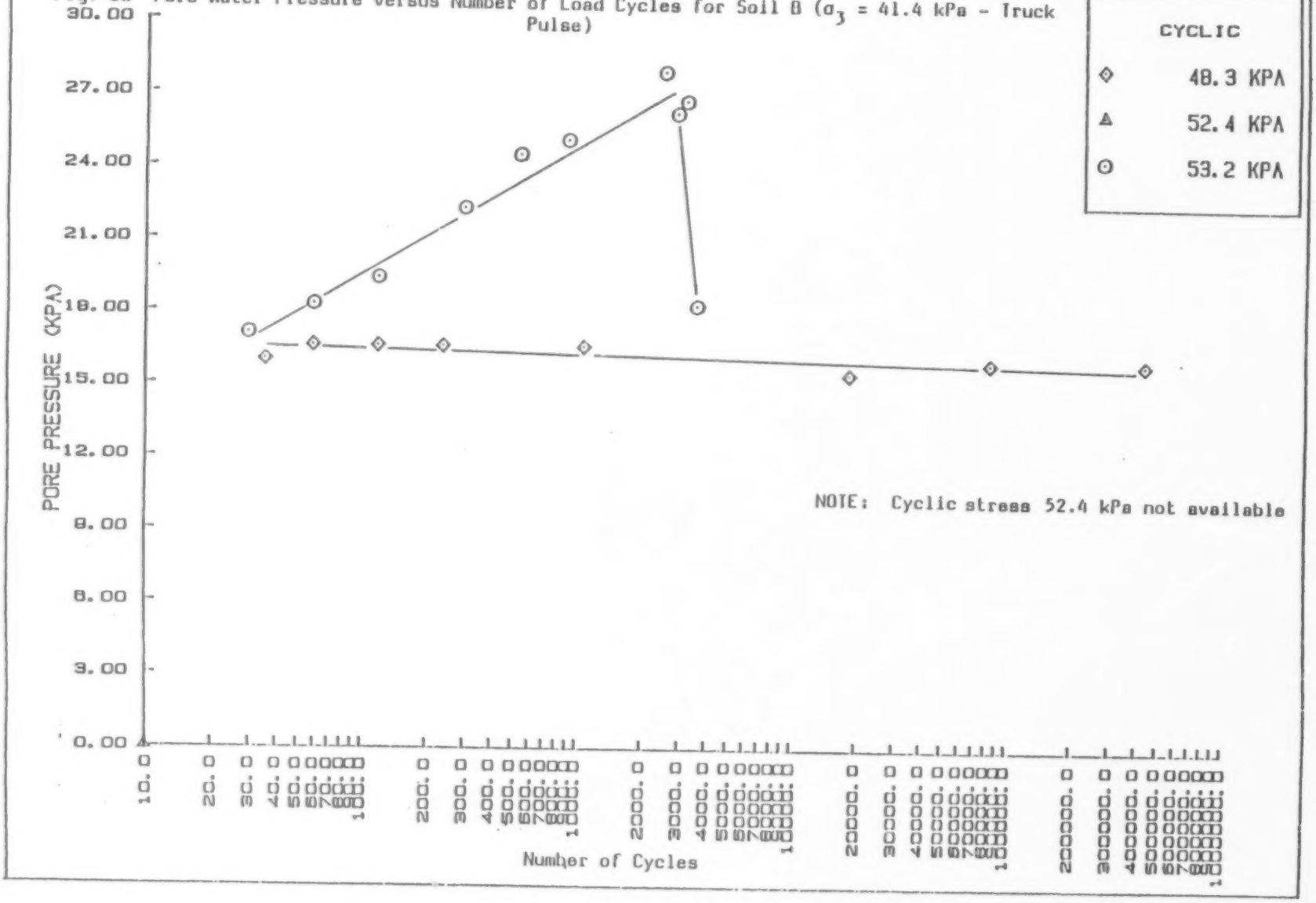


Fig. 60 Pore Water Pressure versus Number of Load Cycles for Soil B (σ_3 = 41.4 kPa - Truck Pulse)



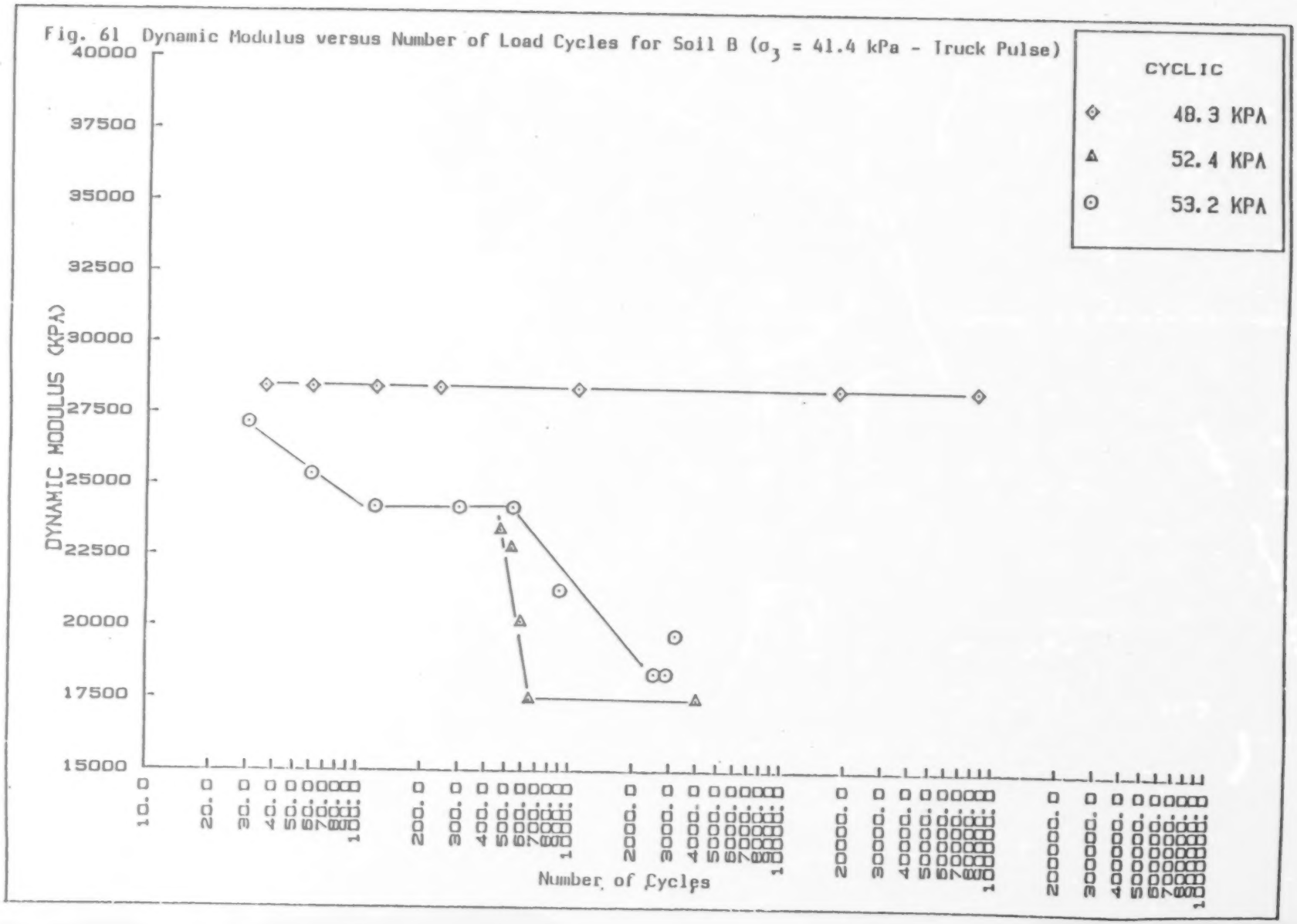


Fig. 61 Dynamic Modulus versus Number of Load Cycles for Soil B (σ_3 = 41.4 kPa - Truck Pulse)
CYCLIC
48.3 KPA
52.4 KPA
53.2 KPA
DYNAMIC MODULUS (KPA)
40000
37500
35000
32500
30000
27500
25000
22500
20000
17500
15000
Number of Cycles

Test Condition	Consolidation Pressure (kPa)	Back Pressure (kPa)	Peak Cyclic Deviator Stress (kPa)	No. of Cycles	Cumulative Strain (%)	Range of Elastic Strain (%)	Range of Dynamic Modulus (MPa)	Range of Pore Pressure (kPa)	Initial Water Content (%)	Final Water Content (%)	Dry Density (Mg/m^3)
undrained	13.8	275.8	19.3	10^6	3.2	.23-.32-.05	8.8-26.5	4.2-6.3	17.0	18.3	1.77
	13.8	275.8	22.1	10^6	3.8	.47-.31-.11	5.0-19.7	6.6-(-1.5)-7.1	17.0	18.3	1.79
	13.8	275.8	27.6	10^6	4.8	.25-.36-.10	11.5-8.6-15.7	5.3-9.3	16.0	19.2	1.78
	13.8	275.8	34.5	500000	7.4	.41-.32-.24	8.8-17.0	N.A.	16.0	19.2	1.77
undrained	27.6	275.8	27.6	370000	0.5	.11-.14	25.3-23.3	8.5-11.6-7.6	17.0	18.0	1.76
	27.6	275.8	27.6 (repeat)	160000	0.6	N.A.	N.A.	8.5-11.6-7.6	17.5	18.3	1.74
	27.6	275.8	34.5	360000	2.0	.10-.07	23.0-40.5	4.7-21.4-16.6	16.0	N.A.	1.77
	27.6	275.8	41.4	180000	3.4	.18-.10	33.1-51.6	N.A.	16.0	18.7	1.77
undrained	41.4	275.8	41.4	800000	3.7	.17-.25-.17	21.0-18.6-17.1	N.A.	17.0	18.0	1.76
	41.4	275.8	44.8	350000	3.9	.17-.25-.17	22.2-17.9-22.2	N.A.	16.0	18.3	1.78
	41.4	275.8	48.0	200000	8.6	.38-.60-.67	13.1-7.9	N.A.	16.0	18.7	1.77
	41.4	275.8	55.2	200000	8.6	.39-.35	14.5-17.1-15.9	N.A.	16.3	19.0	1.78
	41.4	275.8	62.0	36000	10.9	.35-.29	18.1-21.0	23.7-27.4	17.0	19.1	1.75

TABLE 9 Dynamic Triaxial Test Parameters for Joliette Soil (Truck Pulse)

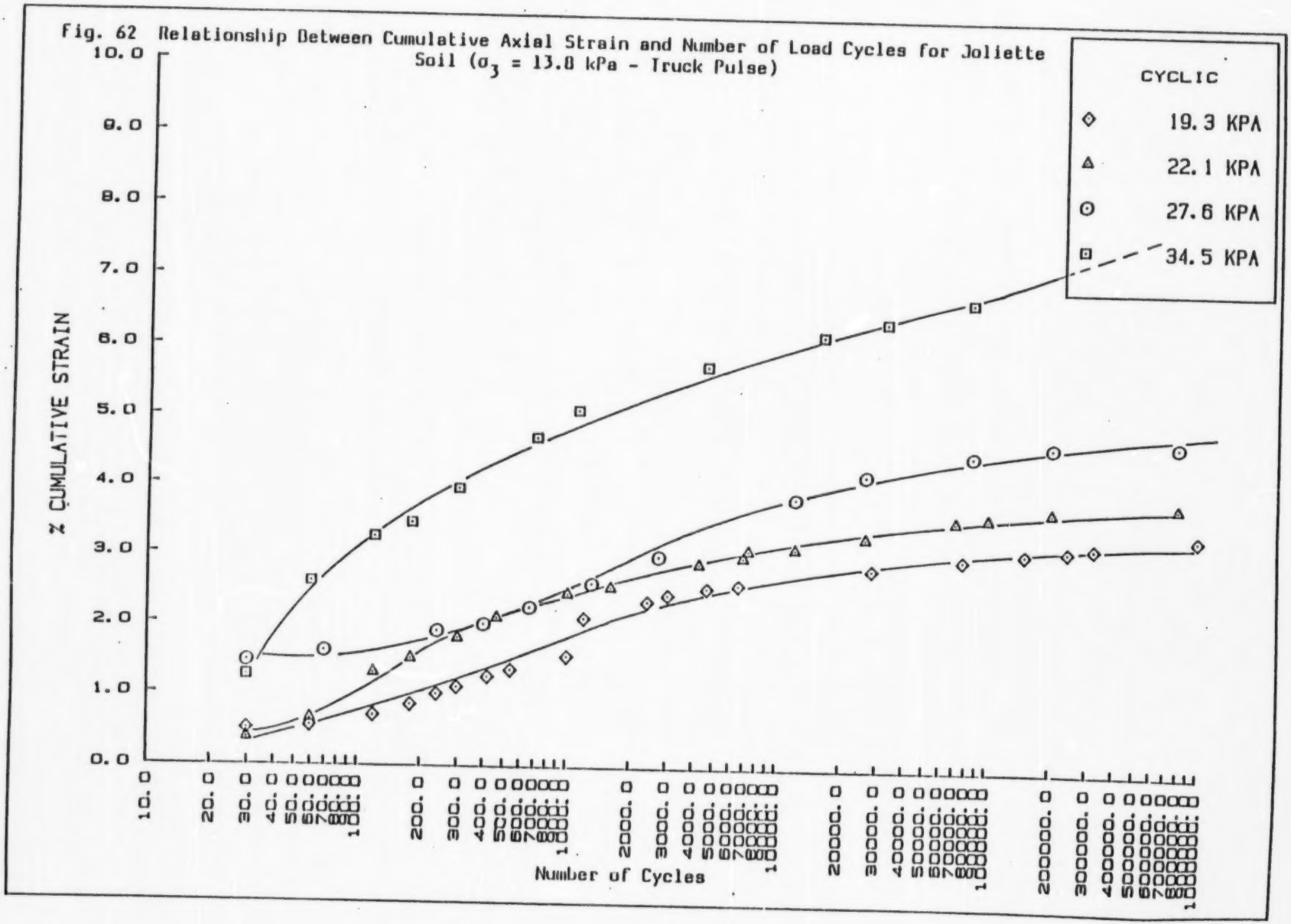
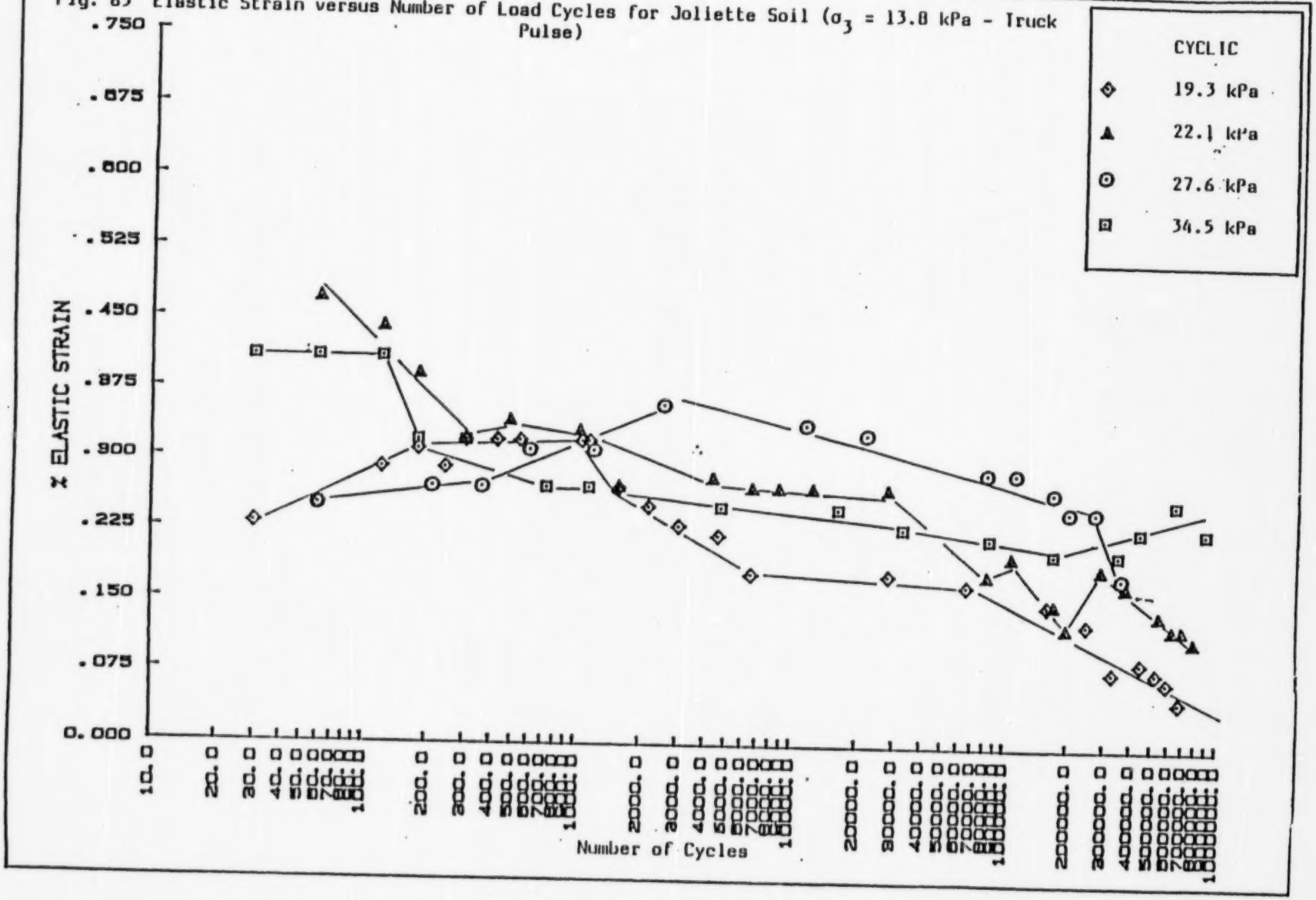


Fig. 62 Relationship Between Cumulative Axial Strain and Number of Load Cycles for Joliette Soil (σ_3 = 13.8 kPa - Truck Pulse)

Fig. 63 Elastic Strain versus Number of Load Cycles for Joliette Soil (σ_3 = 13.8 kPa - Truck Pulse)



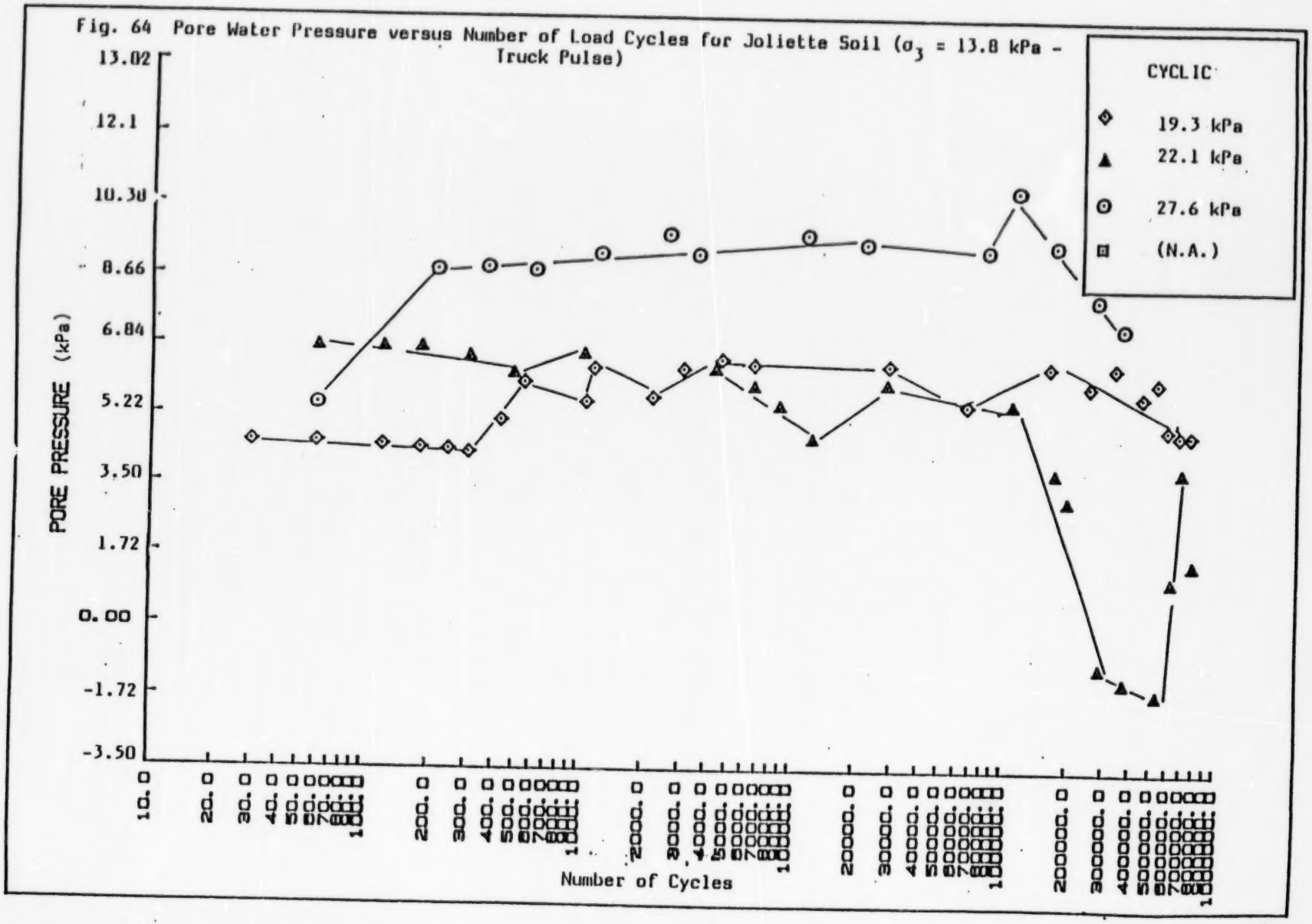


Fig. 64 Pore Water Pressure versus Number of Load Cycles for Joliette Soil (σ_3 = 13.8 kPa - Truck Pulse)
CYCLIC
19.3 kPa
22.1 kPa
27.6 kPa
(N.A.)
PORE PRESSURE (kPa)
13.02
12.1
10.30
8.66
6.84
5.22
3.50
1.72
0.00
-1.72
-3.50
Number of Cycles

Fig. 65 Dynamic Modulus versus Number of Load Cycles for Joliette Soil (σ_3 = 13.8 kPa - Truck Pulse)

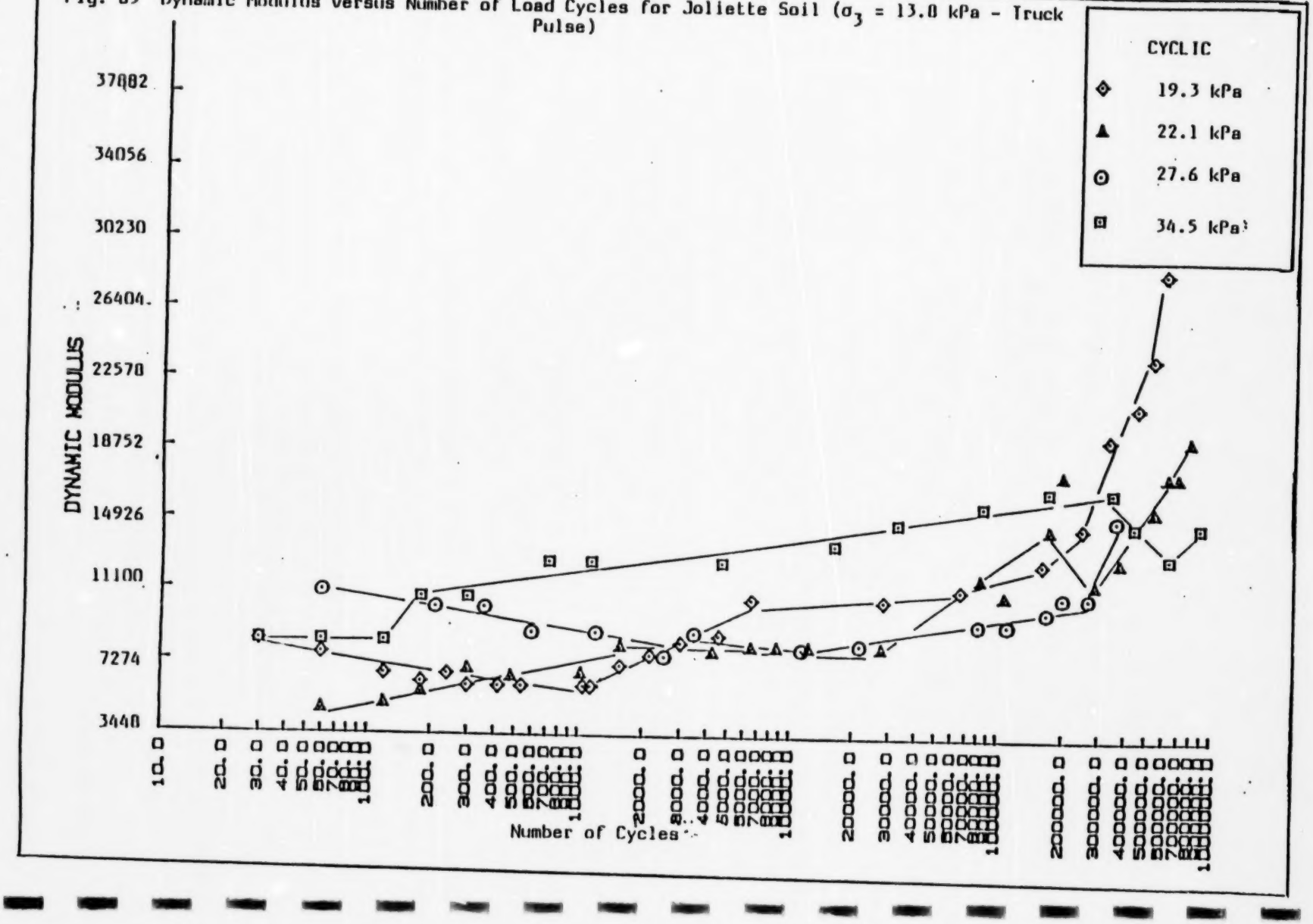


Fig. 66 Relationship Between Cumulative Axial Strain and Number of Load Cycles for Joliette Soil (σ_3 = 27.6 kPa - Truck Pulse)

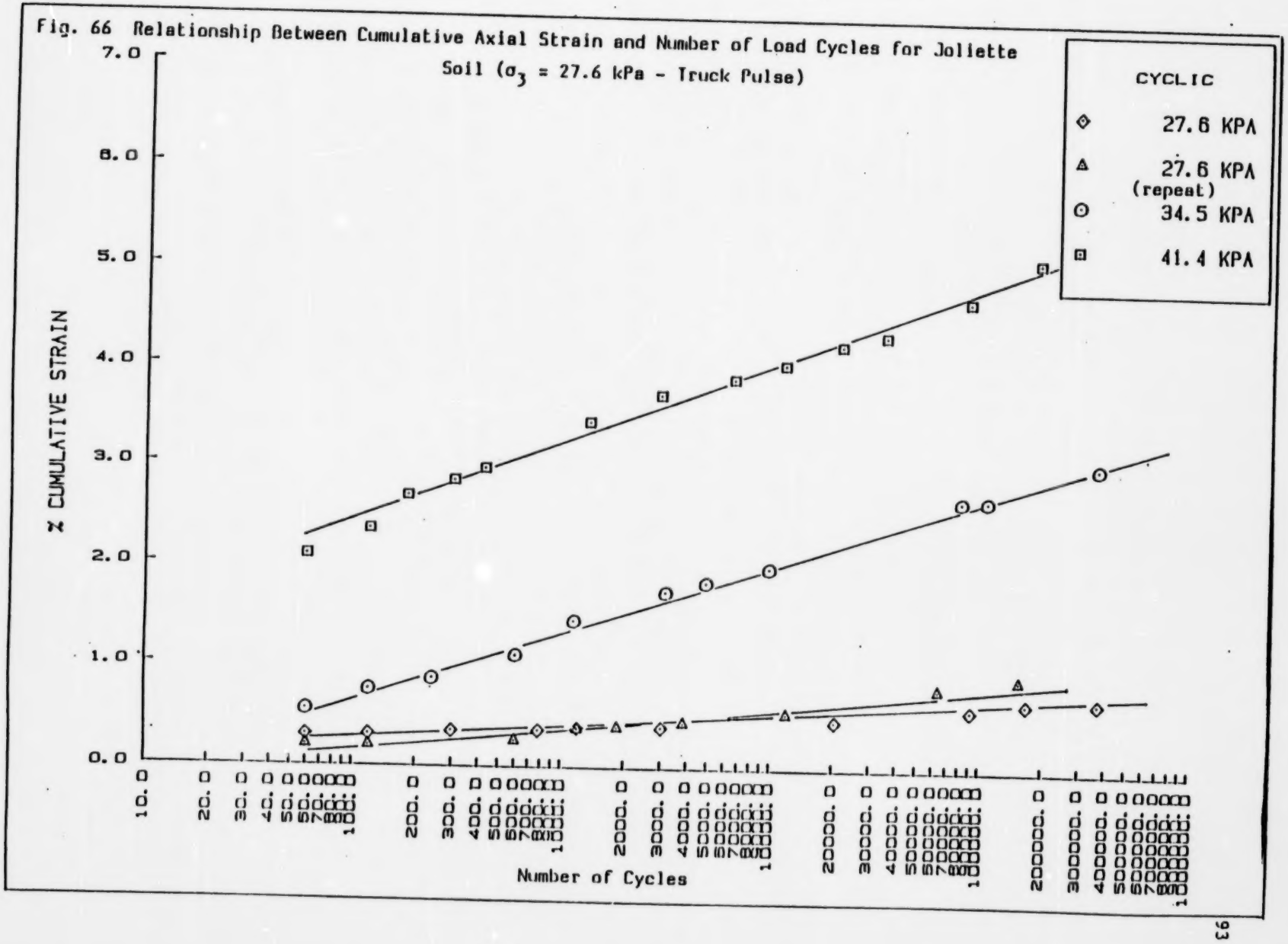
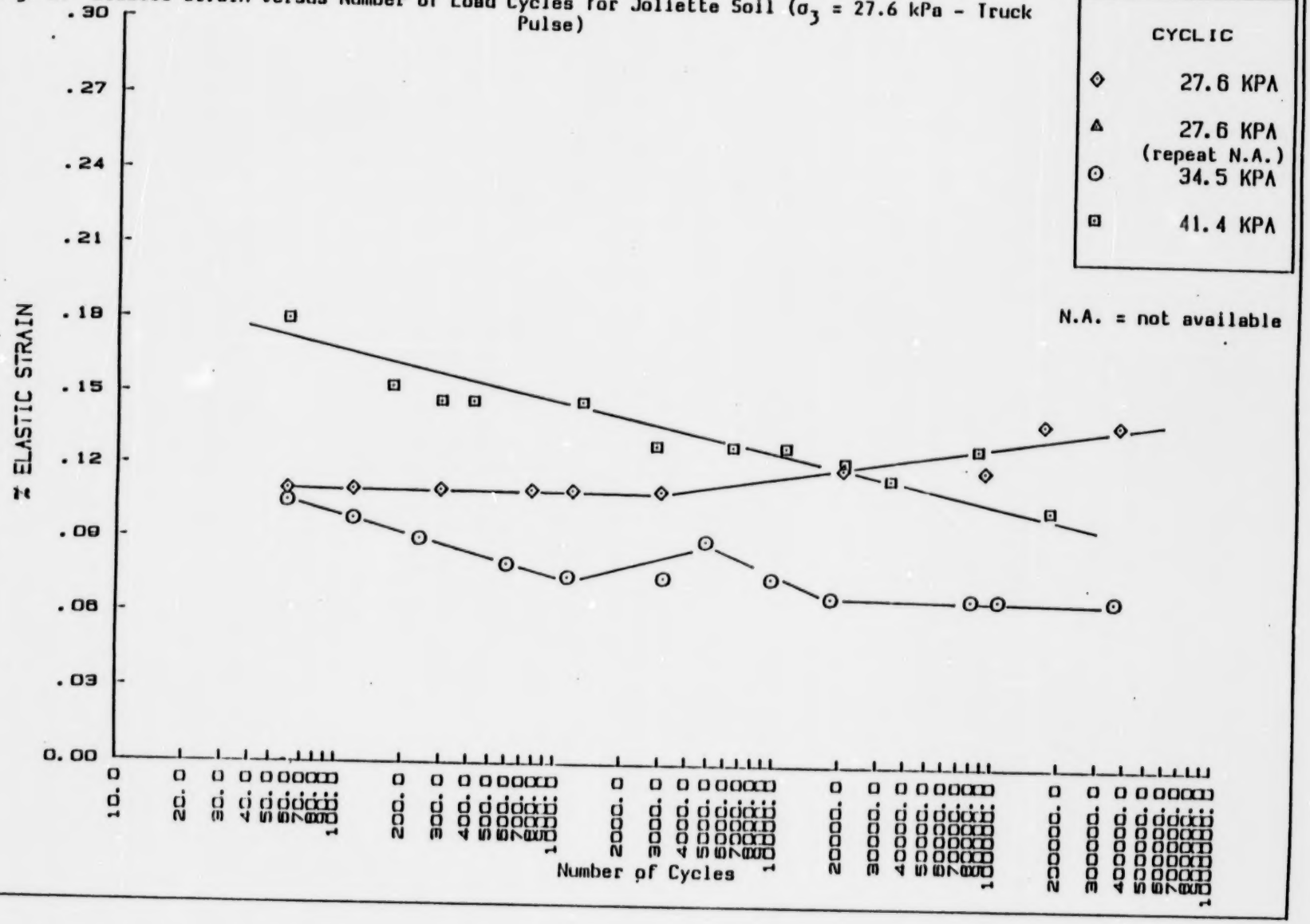


Fig. 67 Elastic Strain versus Number of Load Cycles for Joliette Soil (σ_3 = 27.6 kPa - Truck Pulse)



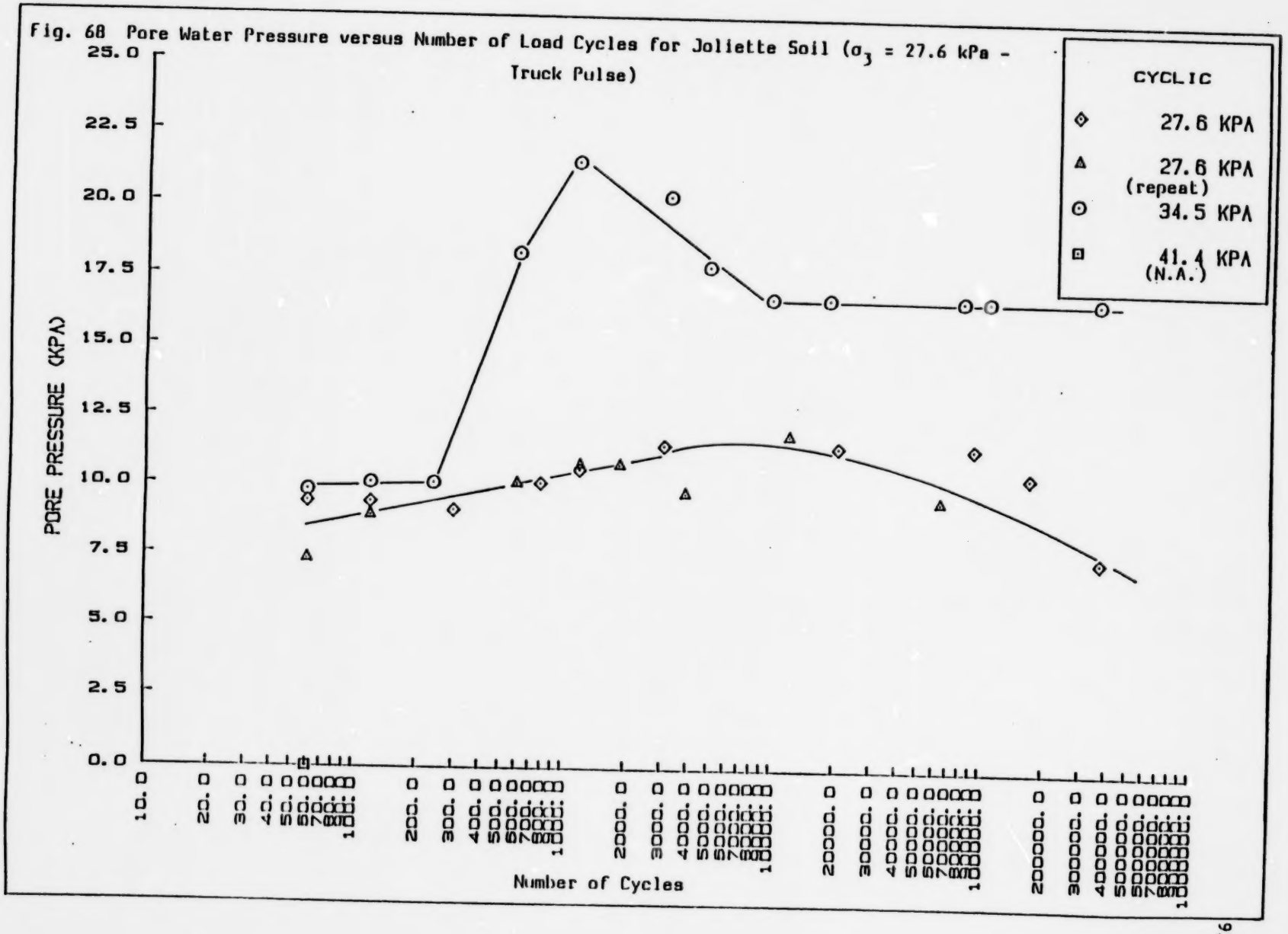
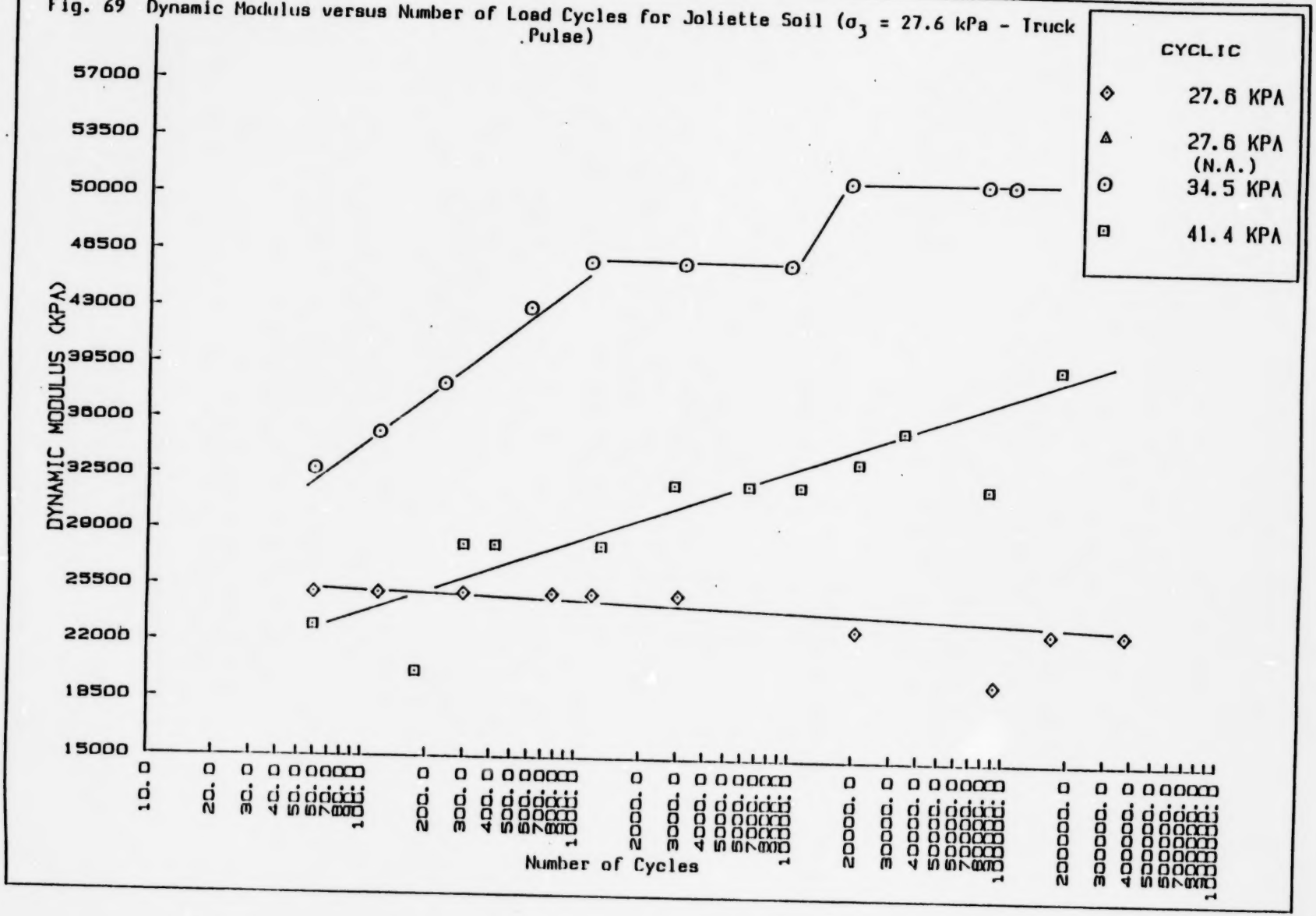


Fig. 68 Pore Water Pressure versus Number of Load Cycles for Joliette Soil (σ_3 = 27.6 kPa - Truck Pulse)
CYCLIC
27.6 KPA
27.6 KPA (repeat)
34.5 KPA
41.4 KPA (N.A.)
PORE PRESSURE (KPA)
25.0
22.5
20.0
17.5
15.0
12.5
10.0
7.5
5.0
2.5
0.0
Number of Cycles

Fig. 69 Dynamic Modulus versus Number of Load Cycles for Joliette Soil (σ_3 = 27.6 kPa - Truck Pulse)



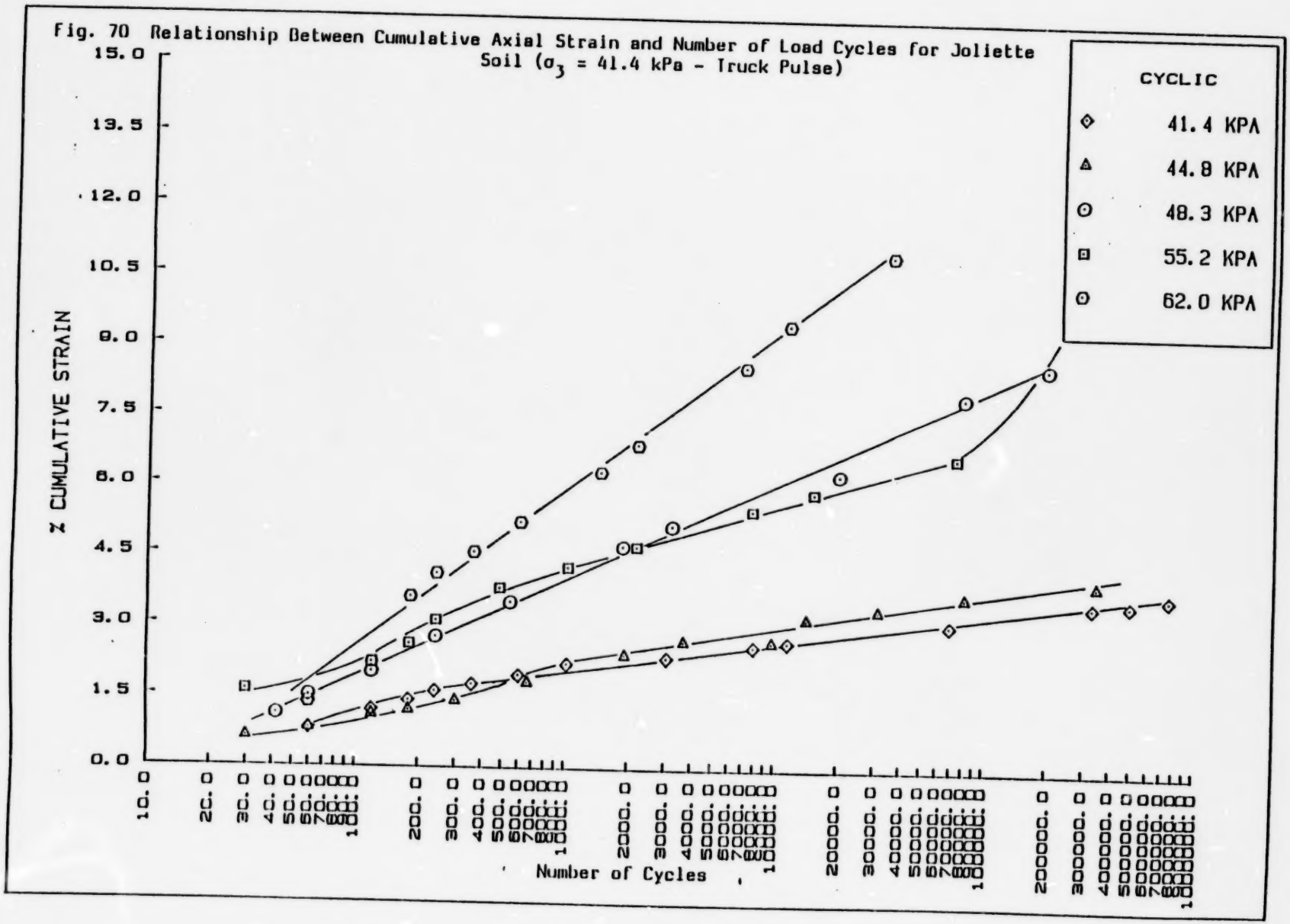


Fig. 70 Relationship Between Cumulative Axial Strain and Number of Load Cycles for Joliette Soil (σ_3 = 41.4 kPa - Truck Pulse)
CYCLIC
41.4 KPA
44.8 KPA
48.3 KPA
55.2 KPA
62.0 KPA
% CUMULATIVE STRAIN
15.0
13.5
12.0
10.5
9.0
7.5
6.0
4.5
3.0
1.5
0.0
Number of Cycles

Fig. 71 Elastic Strain versus Number of Load Cycles for Joliette Soil (σ_3 = 41.4 kPa - Truck Pulse)

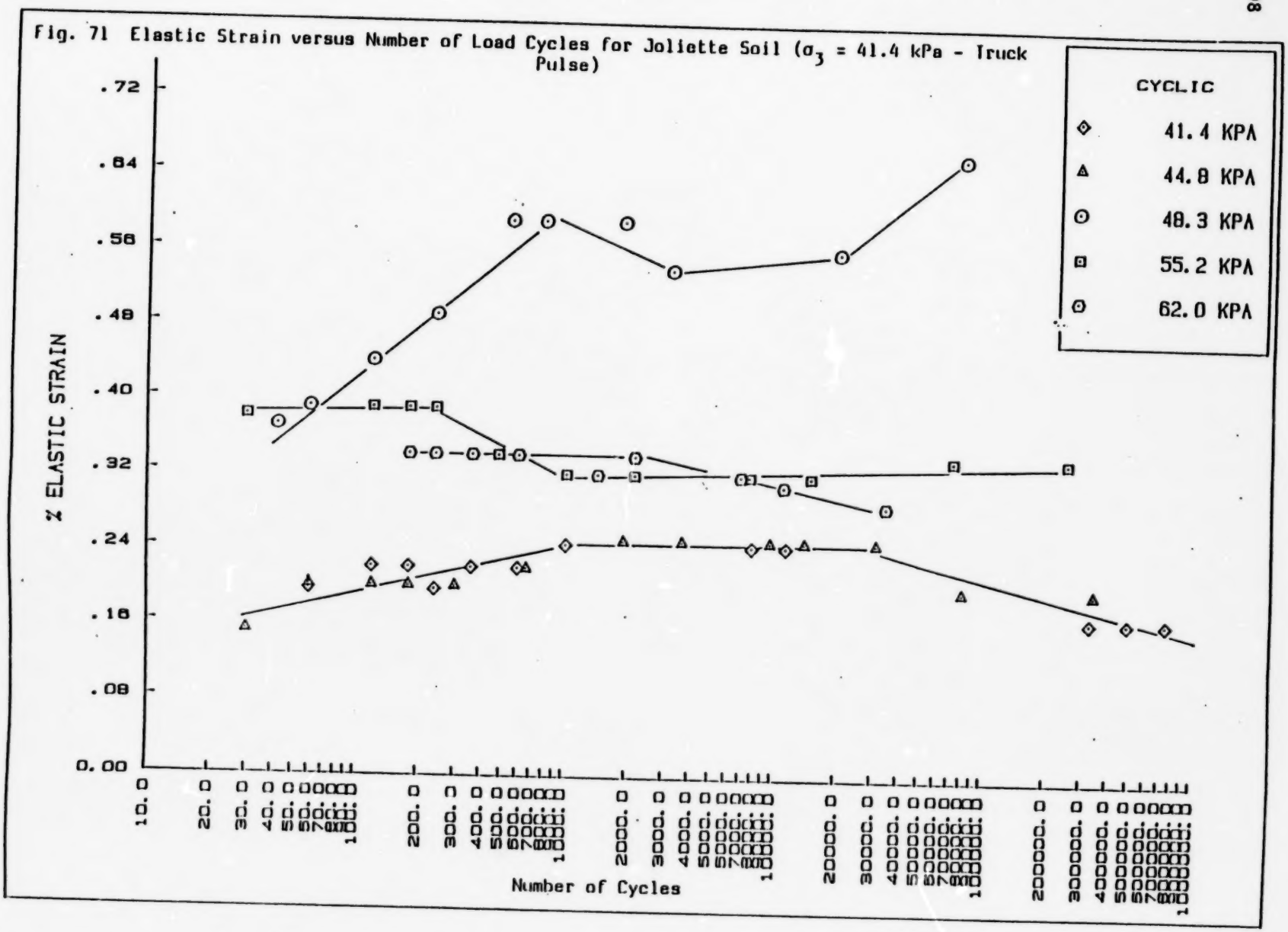


Fig. 72 Pore Water Pressure versus Number of Load Cycles for Joliette Soil (σ_3 = 41.4 kPa - Truck Pulse

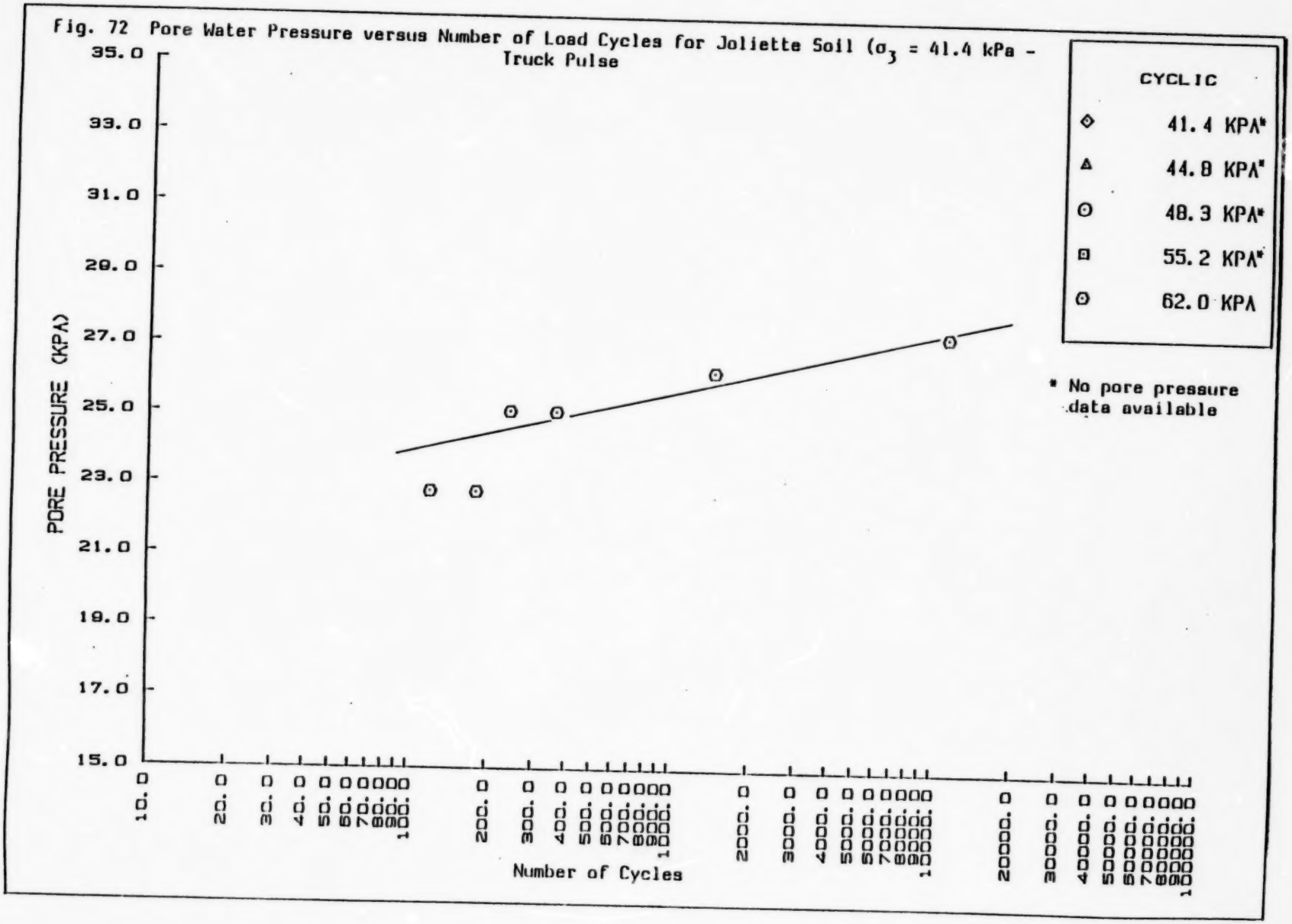
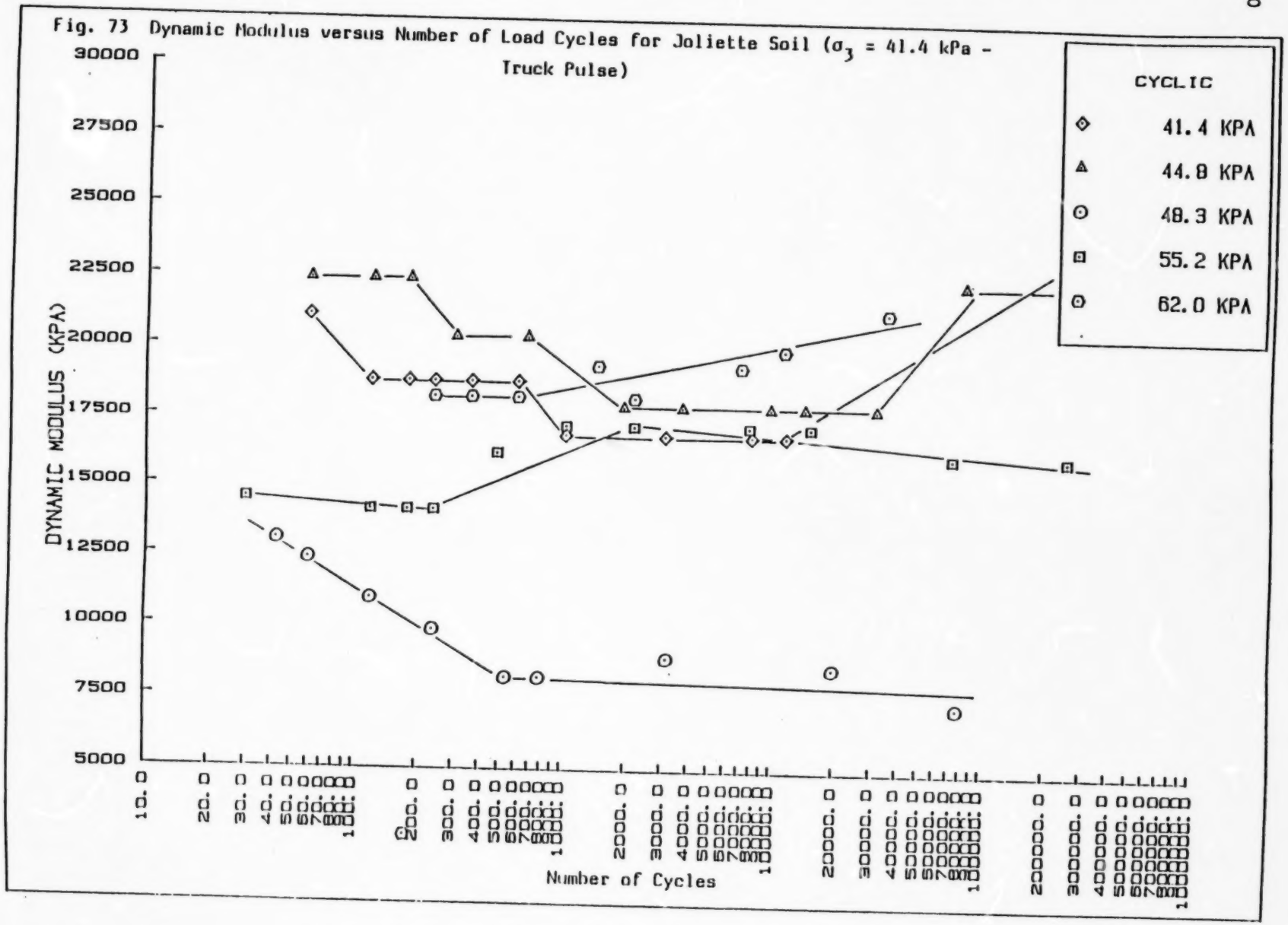


Fig. 73 Dynamic Modulus versus Number of Load Cycles for Joliette Soil (σ_3 = 41.4 kPa - Truck Pulse)



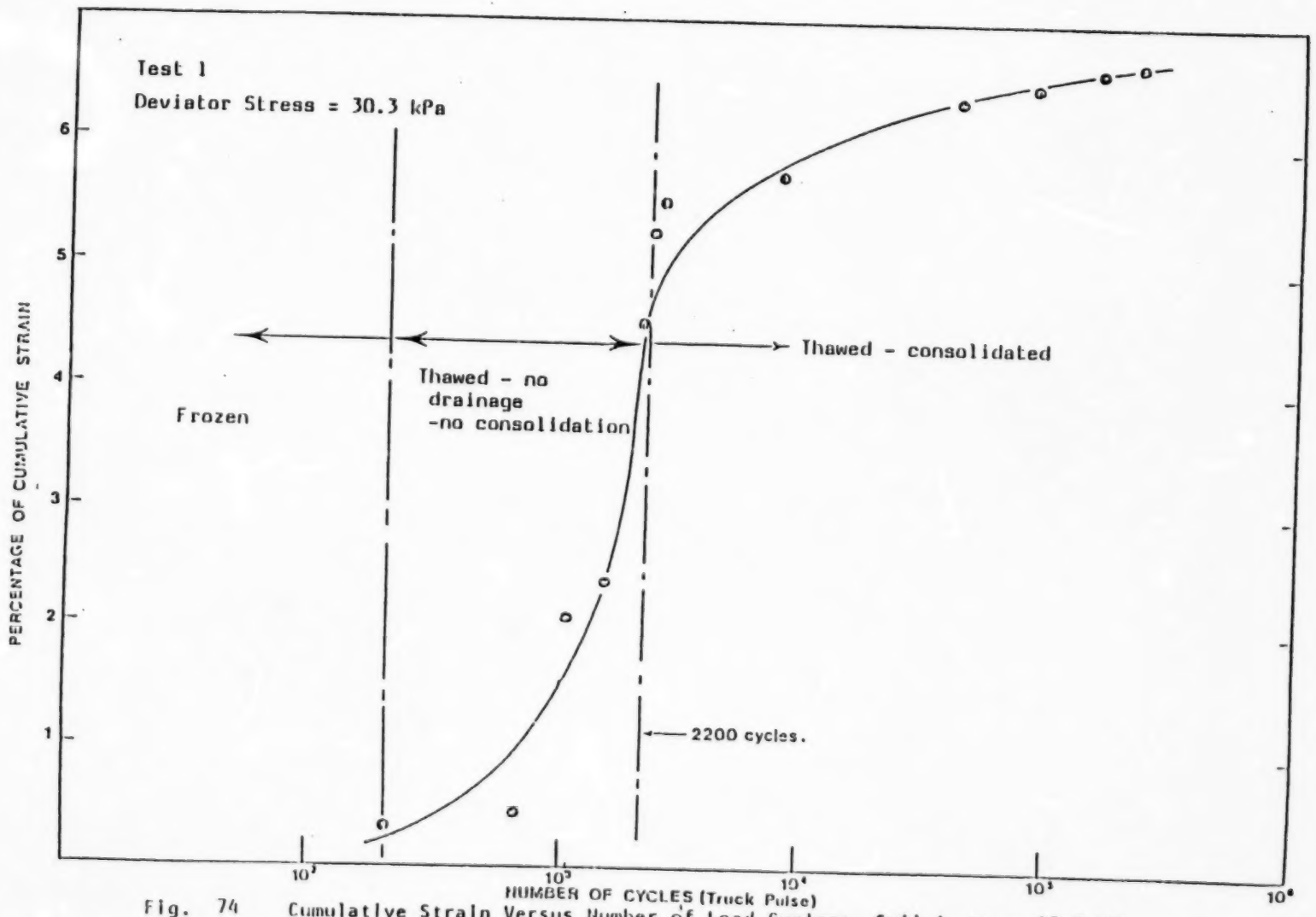


Fig. 74 Cumulative Strain Versus Number of Load Cycles: Soil A, σ_3 = 13.8 kPa

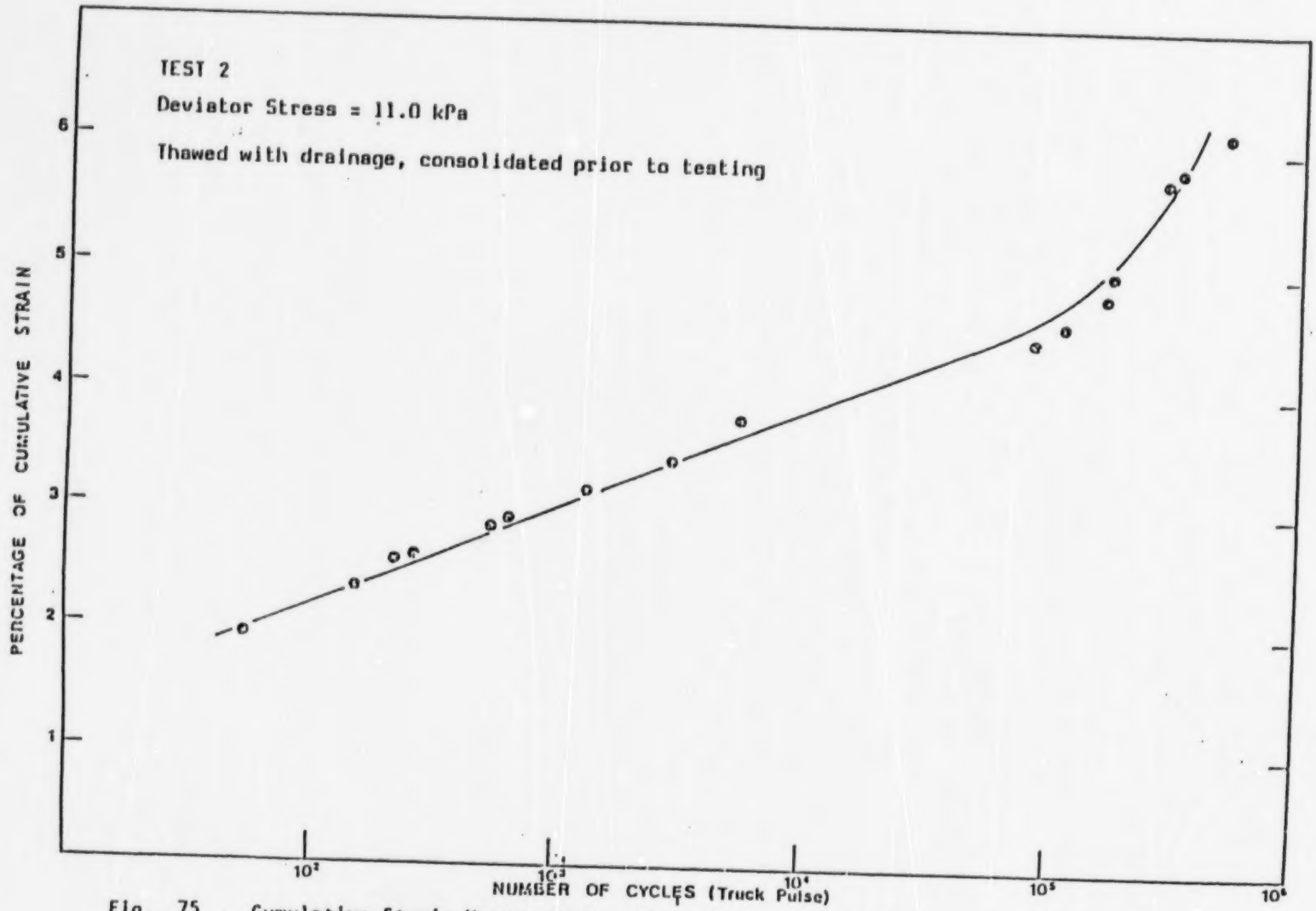


Fig. 75 . Cumulative Strain Versus Number of Load Cycles. Soil Type A. σ_3 = 13.8 kPa

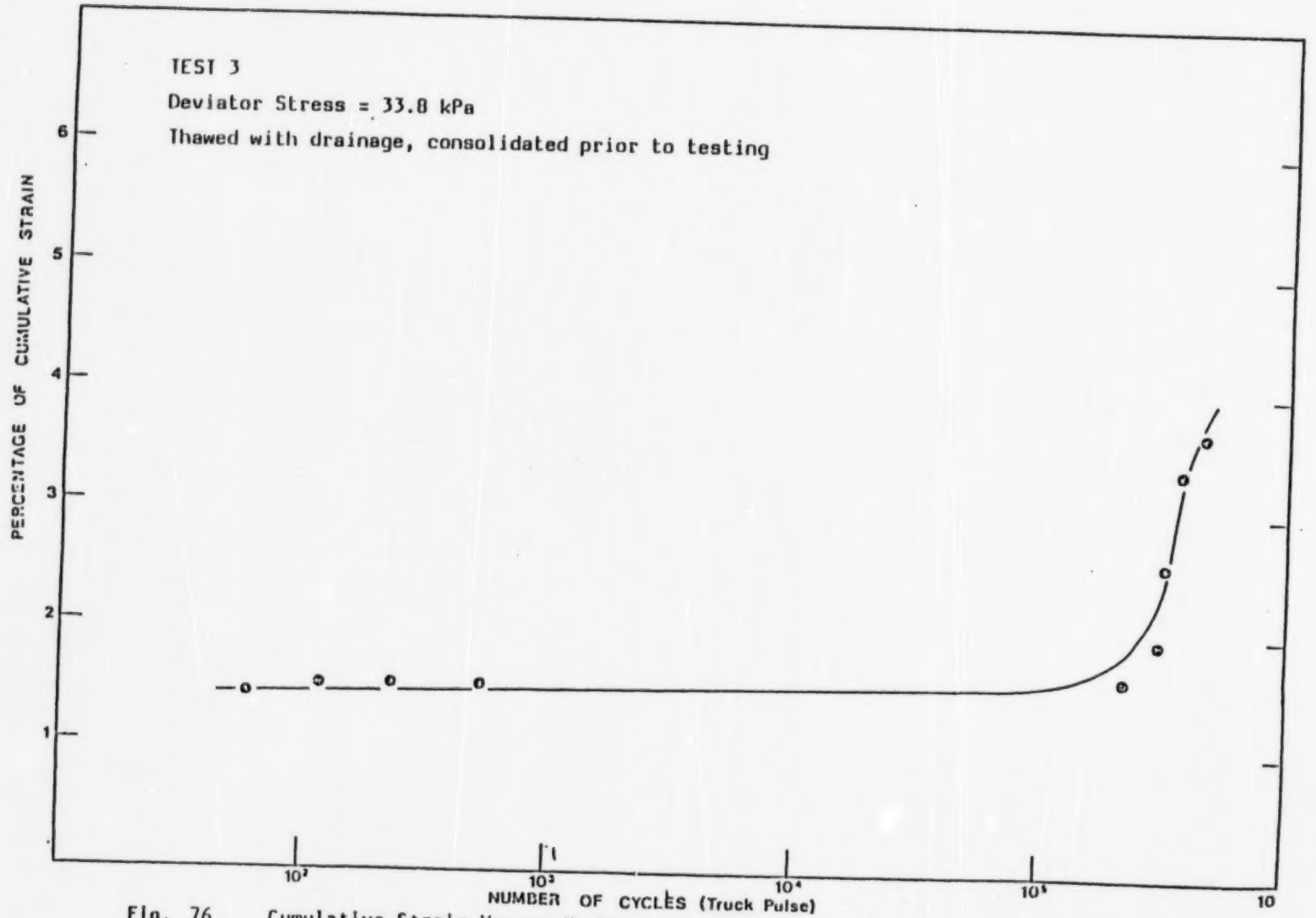


Fig. 76 Cumulative Strain Versus Number of Load Cycles, Soil A, σ_3 = 13.8 kPa

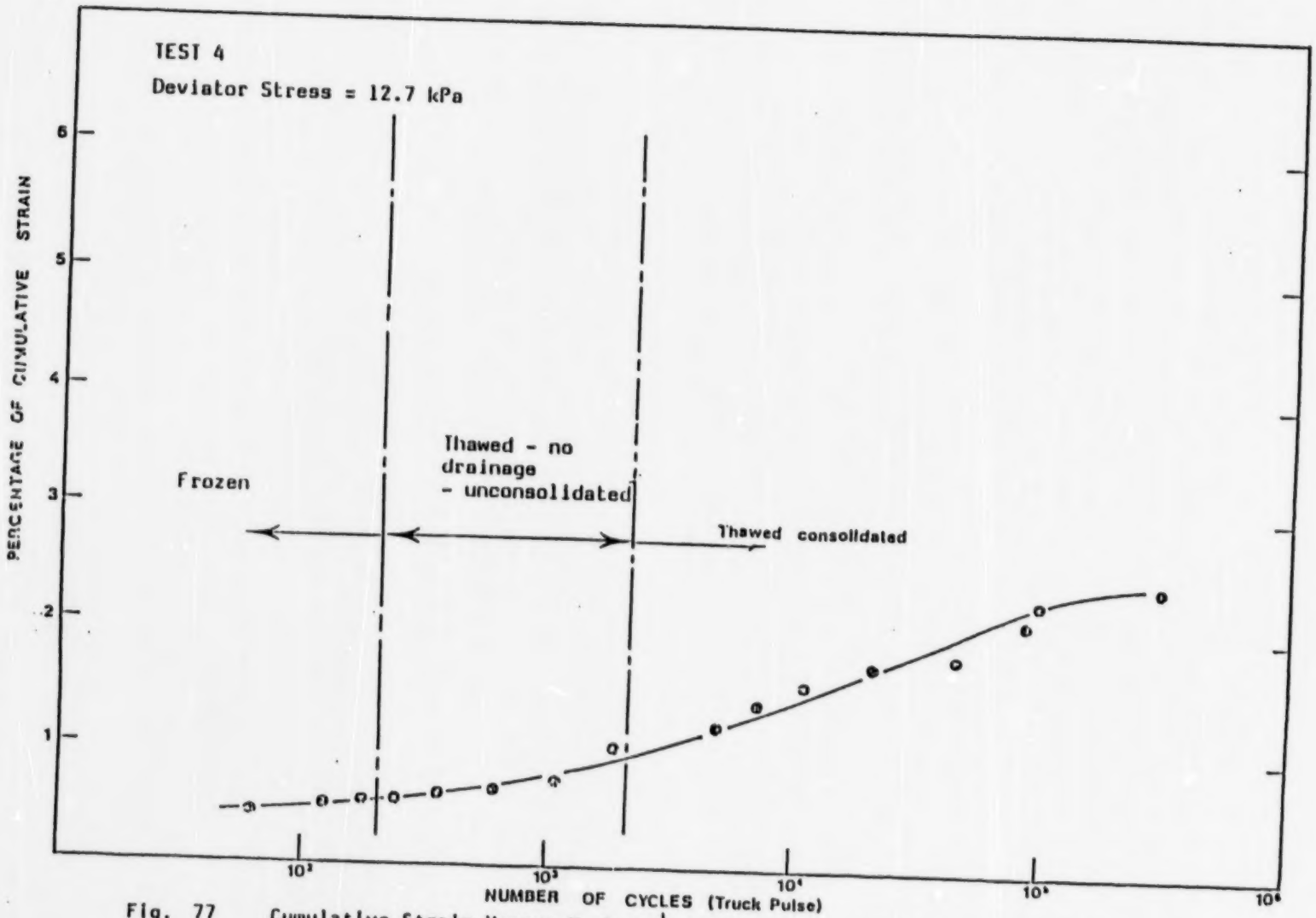


Fig. 77 Cumulative Strain Versus Number of Load Cycles, Soil A, σ_3 = 13.8 kPa

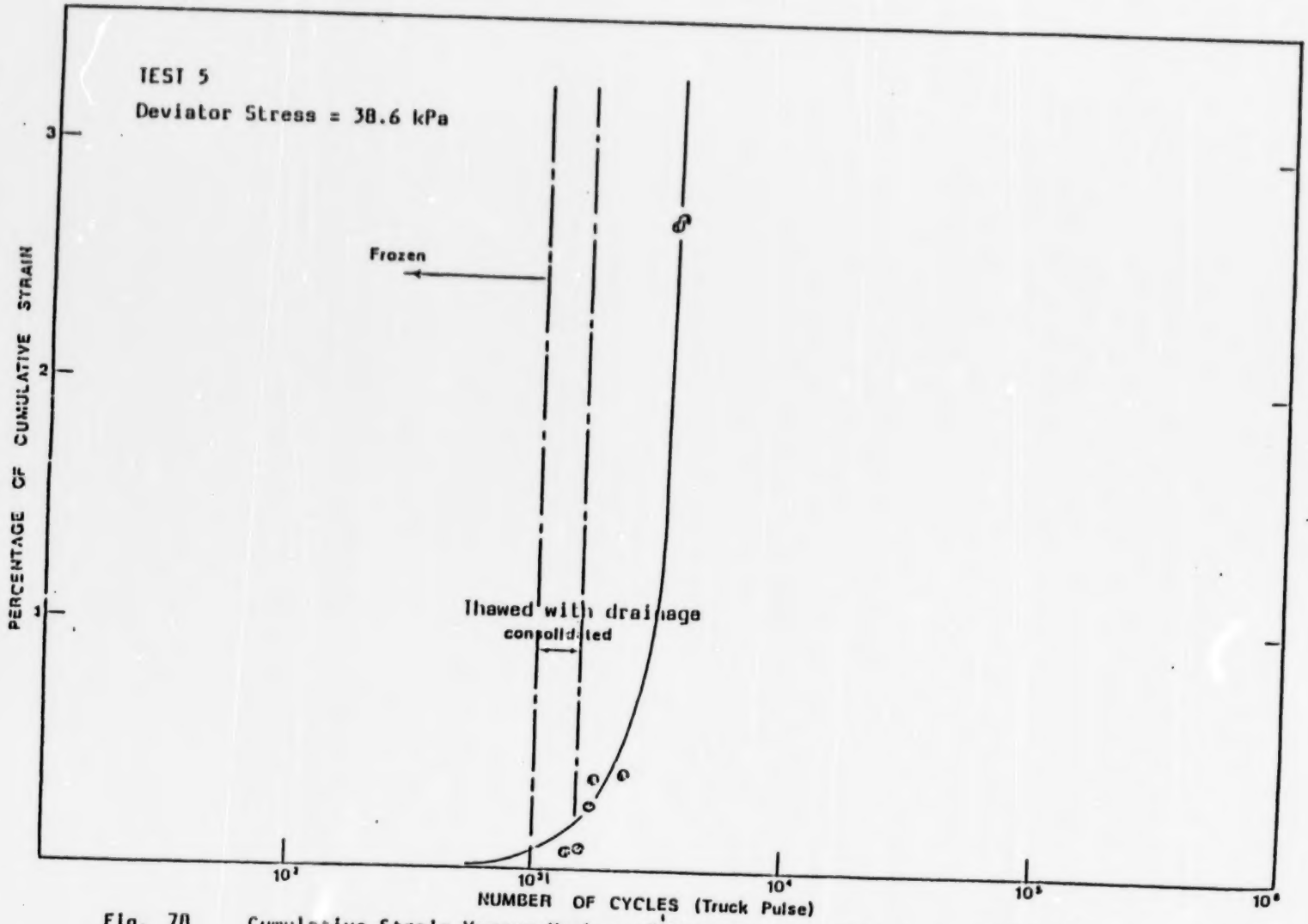
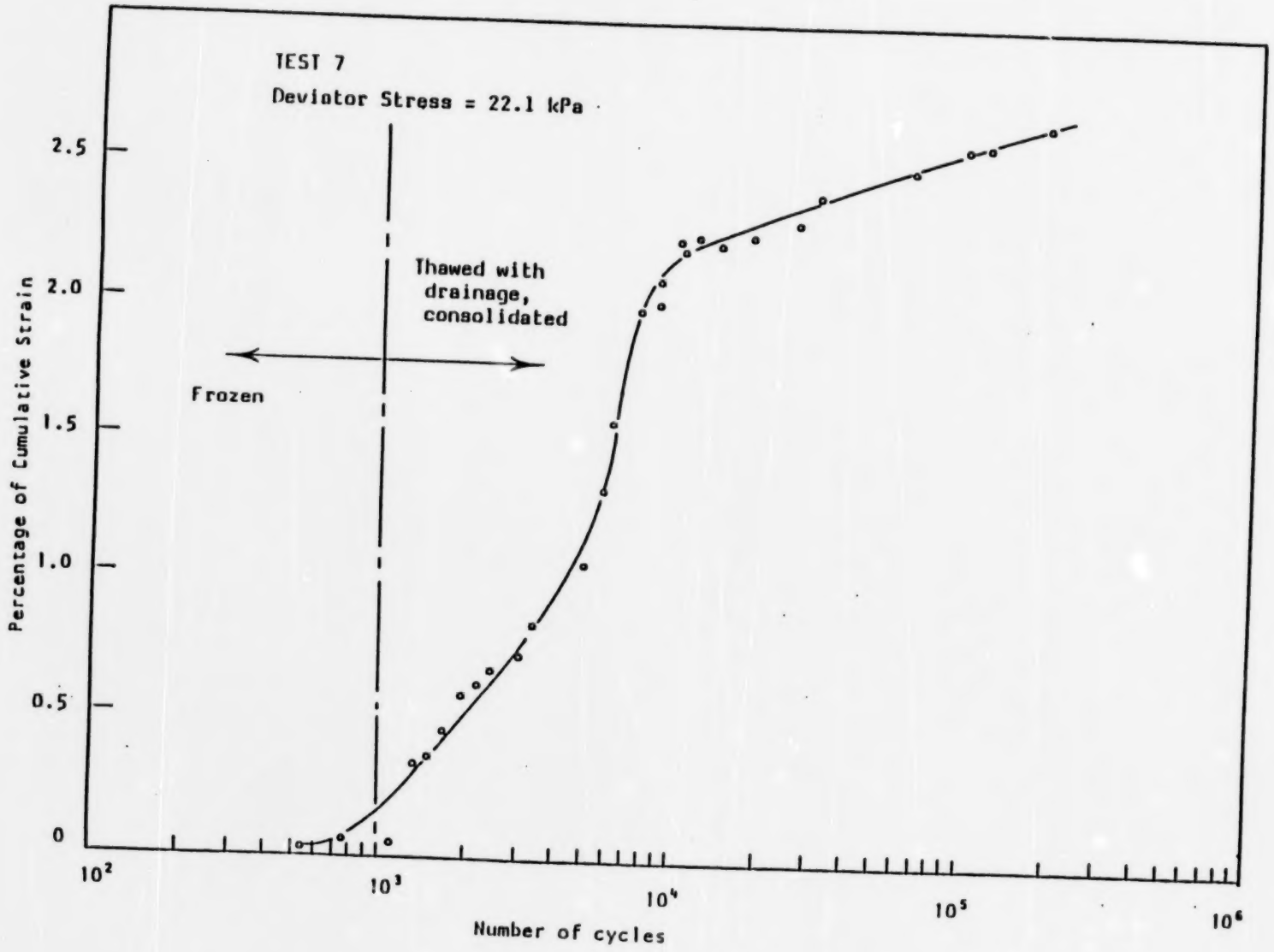


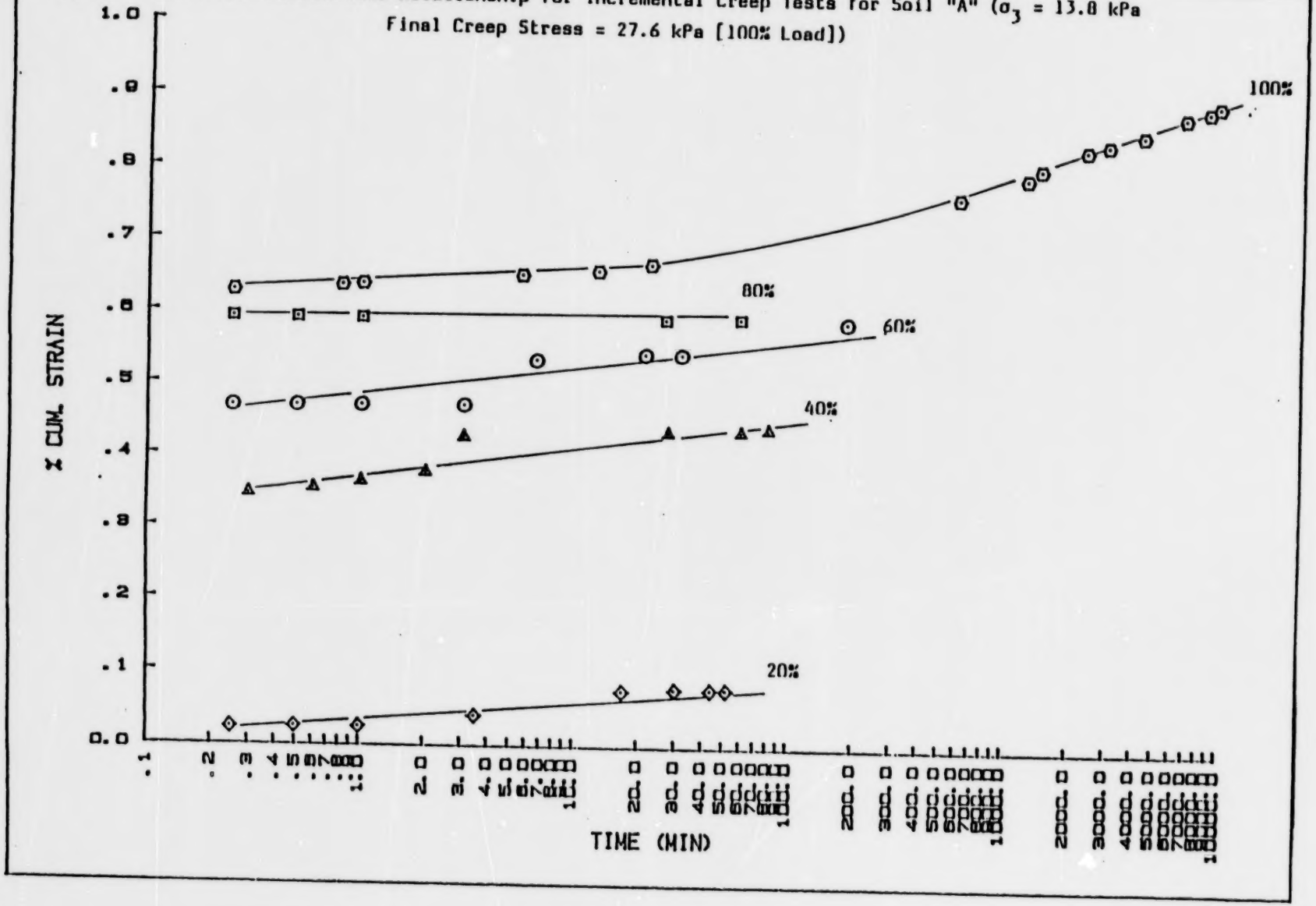
Fig. 78 Cumulative Strain Versus Number of Load Cycles, Soil A, σ_3 = 13.8 kPa


TEST 7
Deviator Stress = 22.1 kPa
Thawed with drainage, consolidated
Frozen
Percentage of Cumulative Strain
2.5
2.0
1.5
1.0
0.5
0
10^2
10^3
10^4
10^5
10^6
Number of cycles

Consolidation Pressure (kPa)	Back Pressure (kPa)	Final Creep Stress (kPa)'	% of Final Creep Stress	% Cumulative Strain	Time (min)	Pore Water Pressure (kPa)	Remarks
13.8	275.8	27.6	20	.08	52	0.64	no failure
			40	.44	79	2.14	
			60	.59	182	3.59	
			80	.60	58	0.83	
			100	.90	9620	1.79	
13.8	275.8	34.5	20	.03	69	1.8	
			40	.09	240	3.9	
			60	.20	767	5.9	
			80	.25	63	7.7	
			100	.57	5540	6.8	failure
13.8	275.8	41.2	20	0.1	55	2.87	
			40	0.86	79	5.93	
			60	1.05	972	6.14	
			80	1.12	303	7.45	
			100	4.12	56	0.41	failure
13.8	275.8	49.4	20	0.03	60	3.2	
			40	.12	75	5.9	
			60	.35	79	7.3	
			80	.54	306	9.5	
			100	3.7	36.5	-1.5	failure

TABLE 10 Summary of Incremental Creep Tests on Soil "A" - σ_3 = 13.8 kPa

Fig. 80 Cumulative Strain-Time Relationship for Incremental Creep Tests for Soil "A" (σ_3 = 13.8 kPa Final Creep Stress = 27.6 kPa [100% Load])



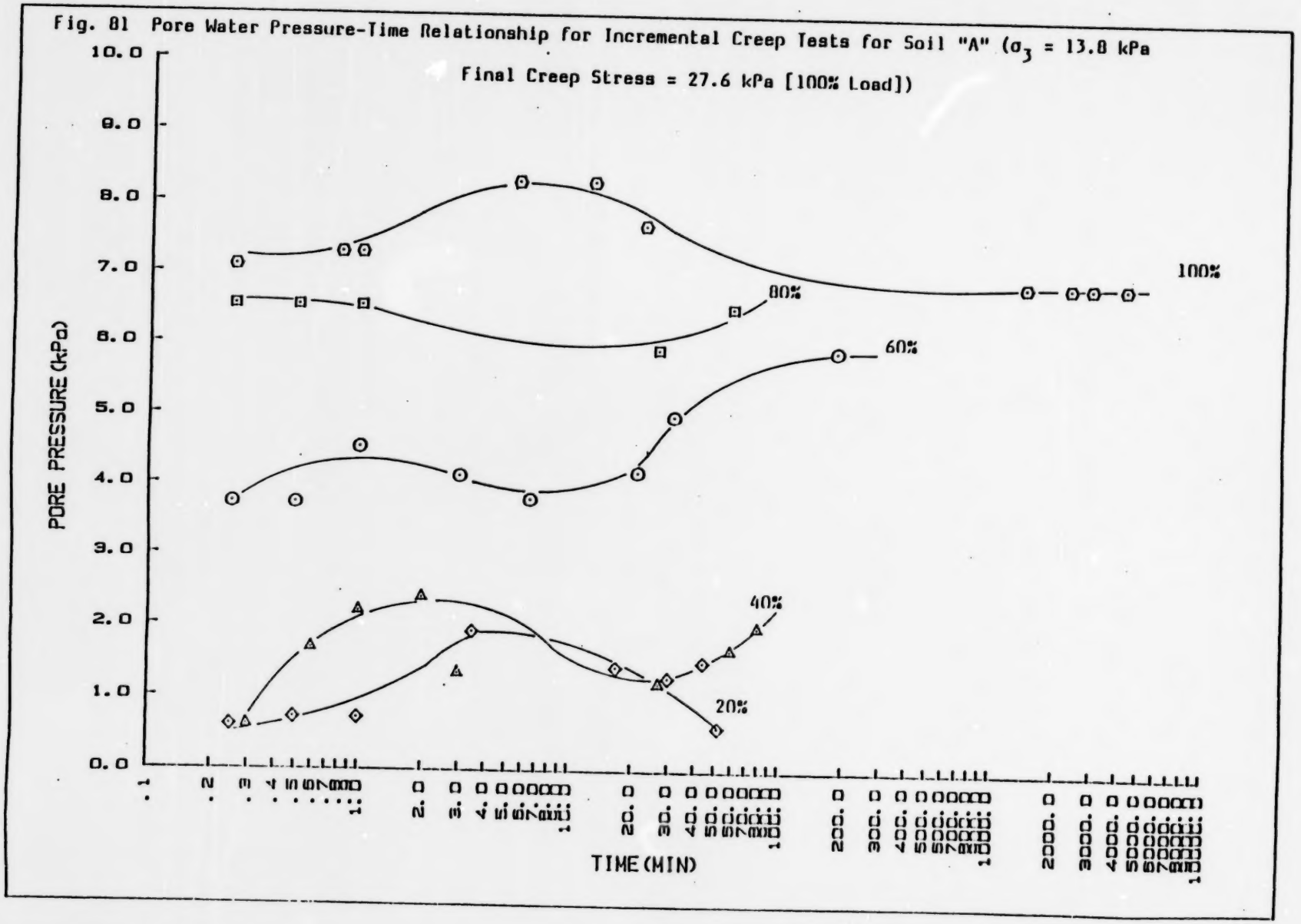


Fig. 81 Pore Water Pressure-Time Relationship for Incremental Creep Tests for Soil "A" (σ_3 = 13.8 kPa
Final Creep Stress = 27.6 kPa [100% Load])
PORE PRESSURE (kPa)
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
100%
80%
60%
40%
20%
TIME (MIN)

Fig. 82 Cumulative Strain-Time Relationship for Incremental Creep Tests Soil "A" (σ_3 = 13.8 kPa
Final Creep Stress = 34.5 kPa [100% Load])

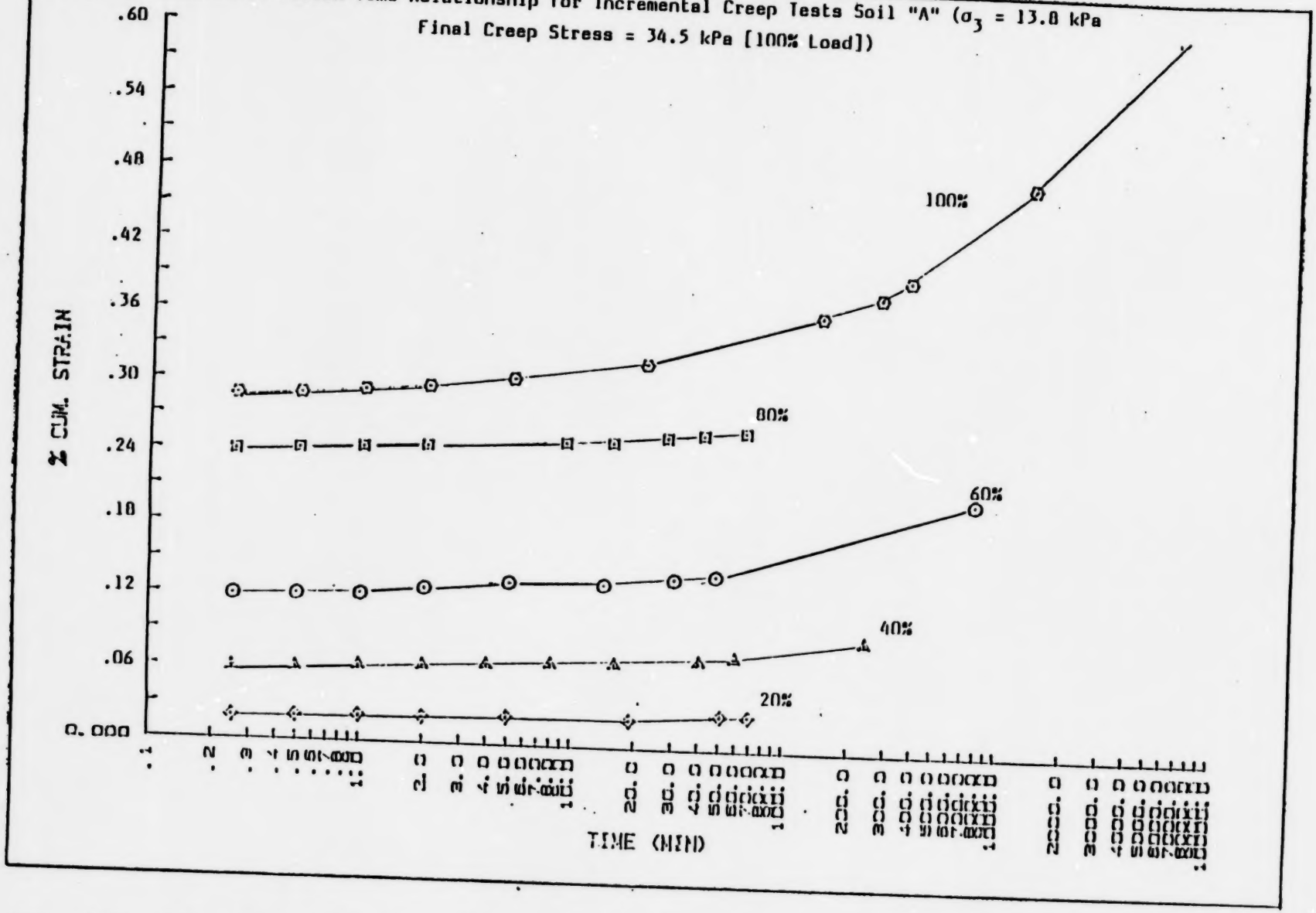


Fig. 83 Pore Water Pressure-Time Relationship for Incremental Creep Test for Soil "A"
(σ_3 = 13.8 kPa, Final Creep Stress = 34.5 kPa [100% Load])

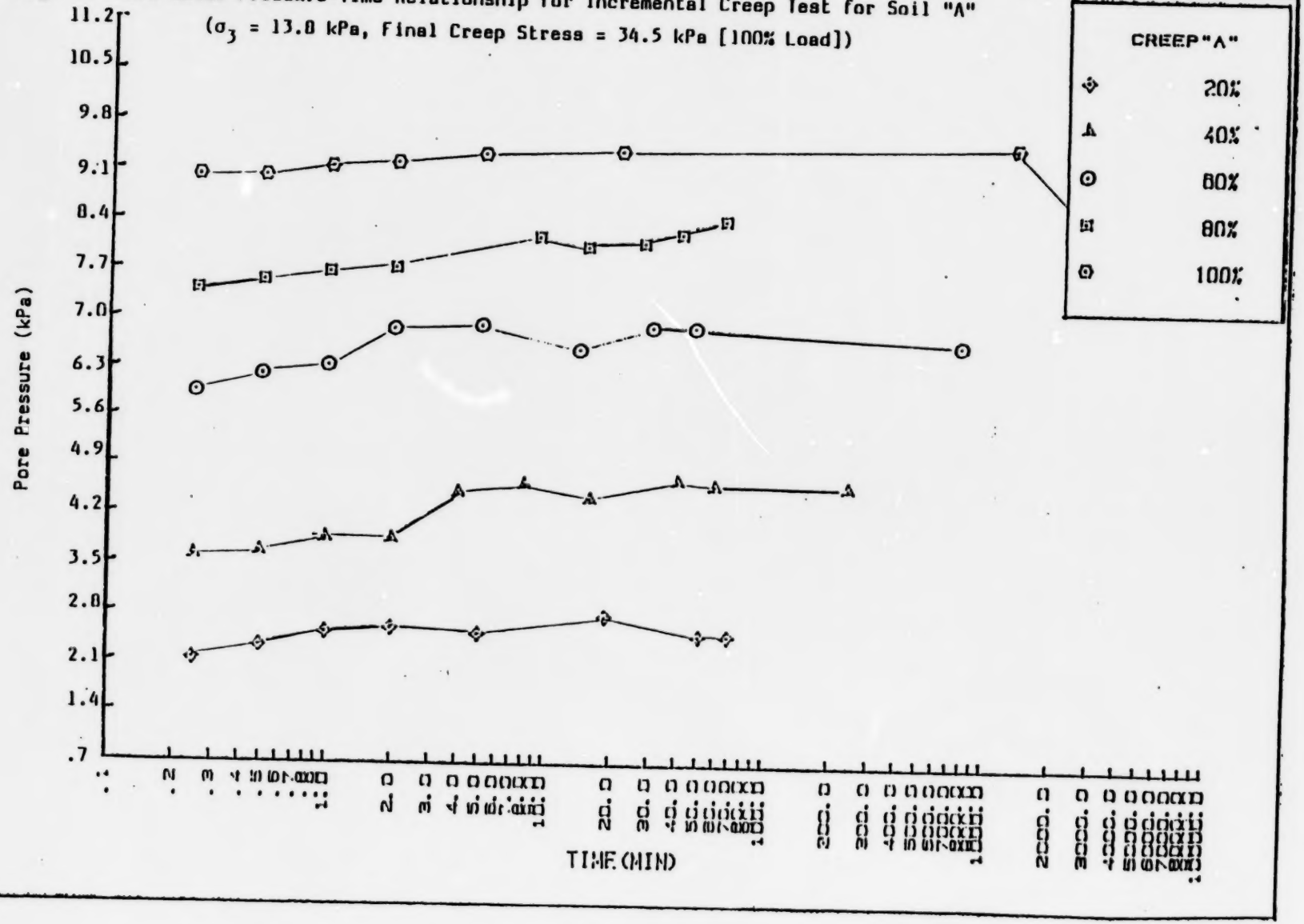
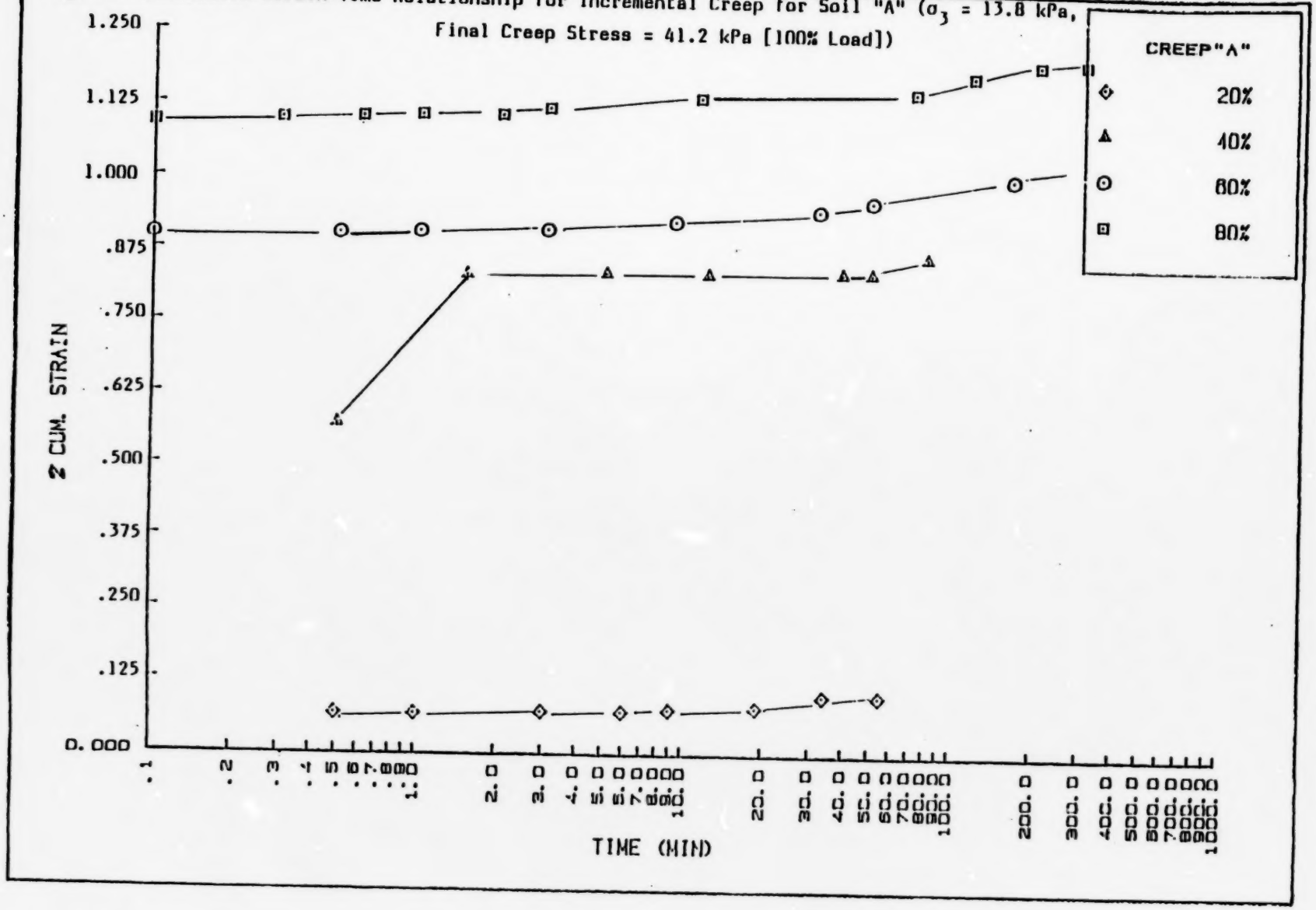


Fig. 84 Cumulative Strain-Time Relationship for Incremental Creep for Soil "A" (σ_3 = 13.8 kPa, Final Creep Stress = 41.2 kPa [100% Load])



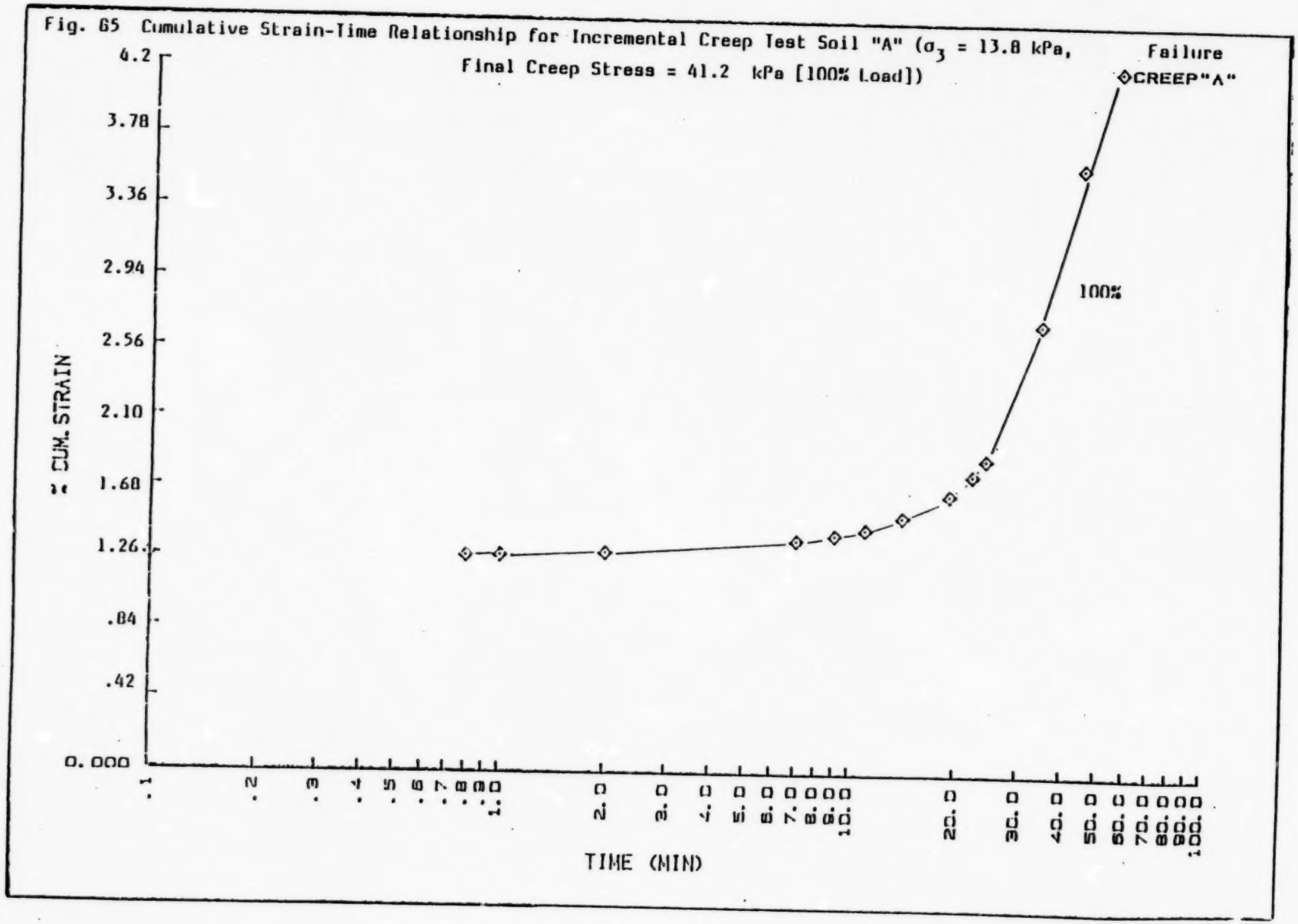
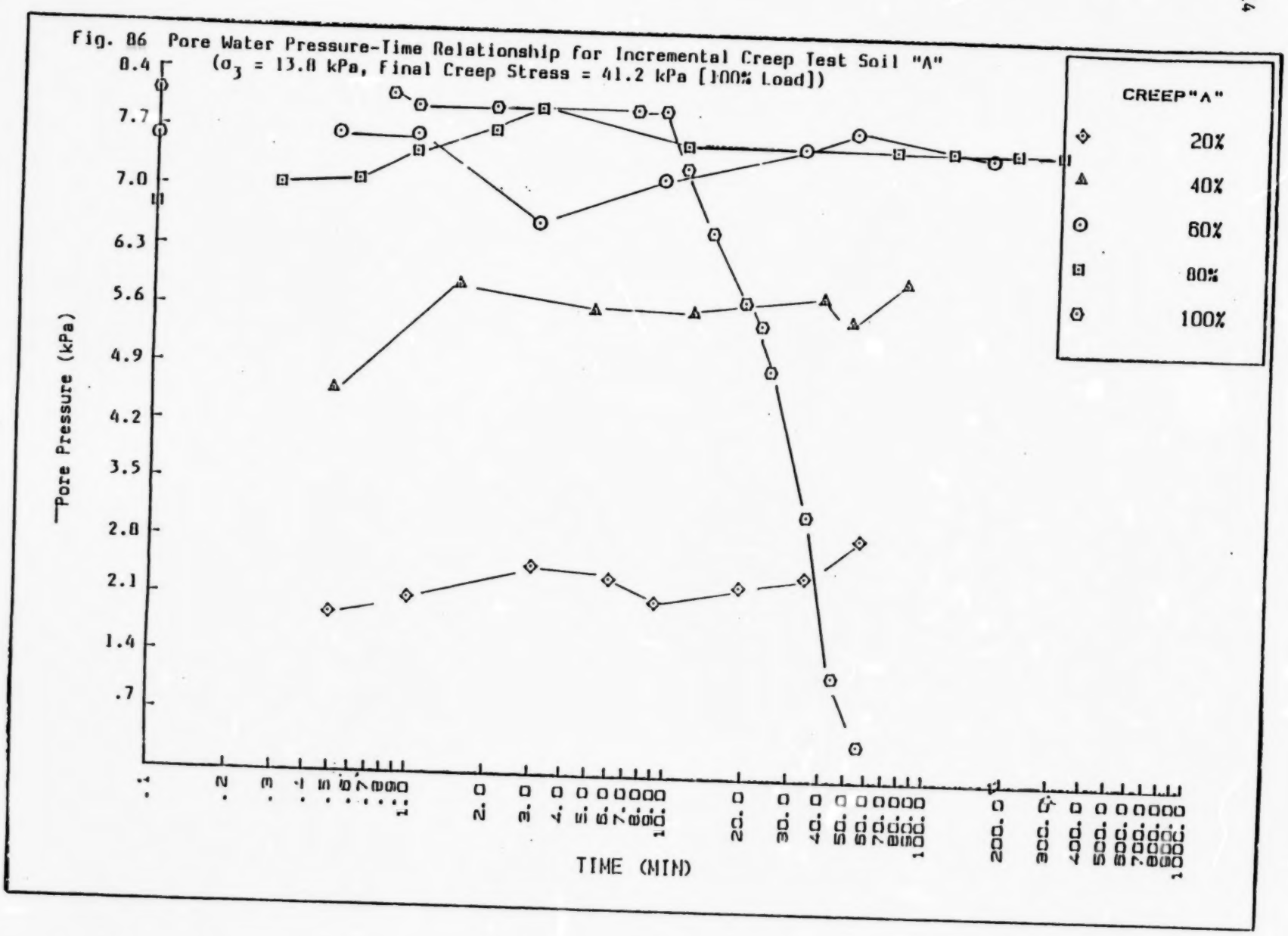


Fig. 65 Cumulative Strain-Time Relationship for Incremental Creep Test Soil "A" (σ_3 = 13.8 kPa,
Final Creep Stress = 41.2 kPa [100% Load])
Failure
CREEP"A"
100%
% CUM. STRAIN
4.2
3.78
3.36
2.94
2.56
2.10
1.68
1.26
.84
.42
0.000
TIME (MIN)

Fig. 86 Pore Water Pressure-Time Relationship for Incremental Creep Test Soil "A" (σ_3 = 13.8 kPa, Final Creep Stress = 41.2 kPa [100% Load])



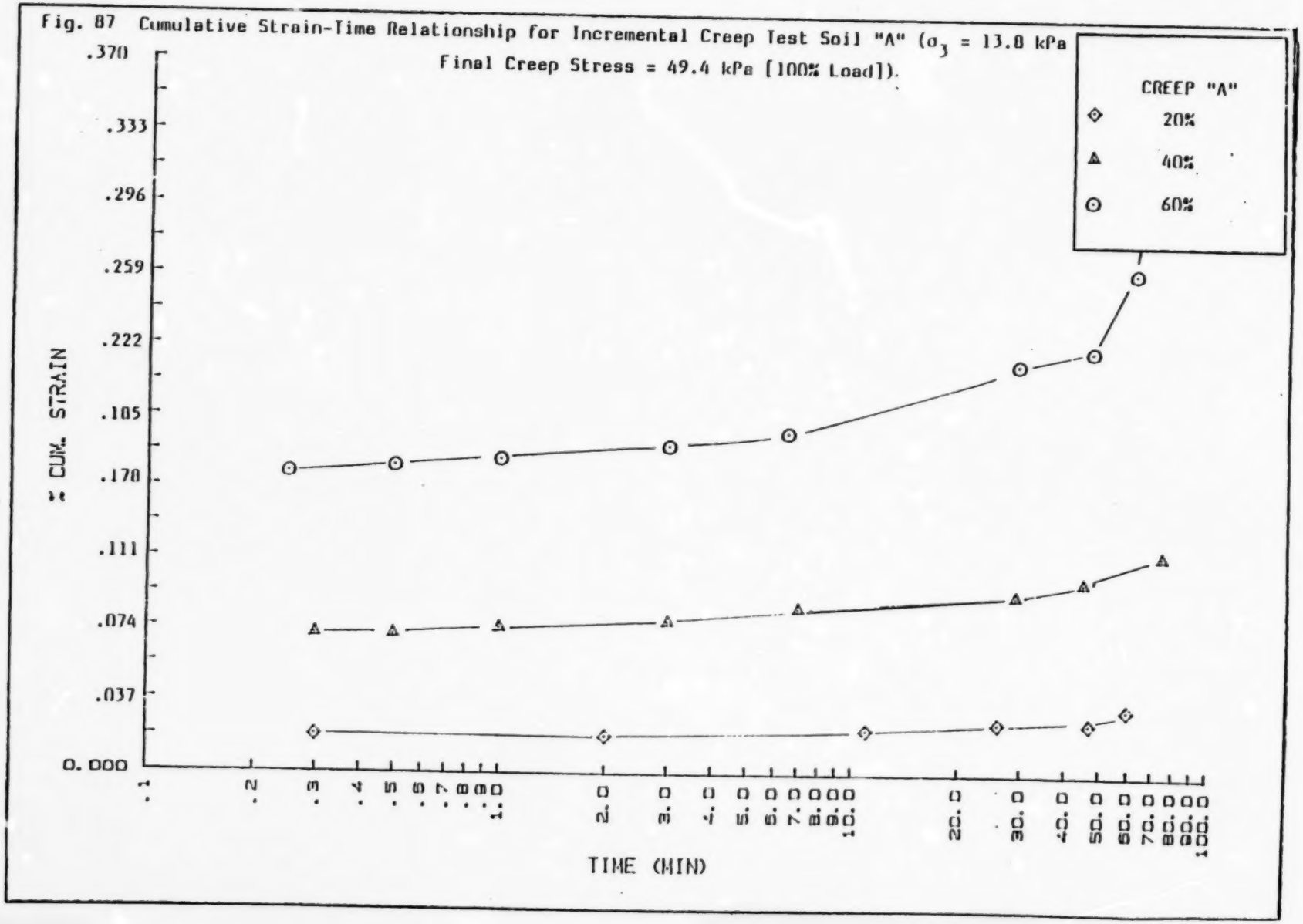
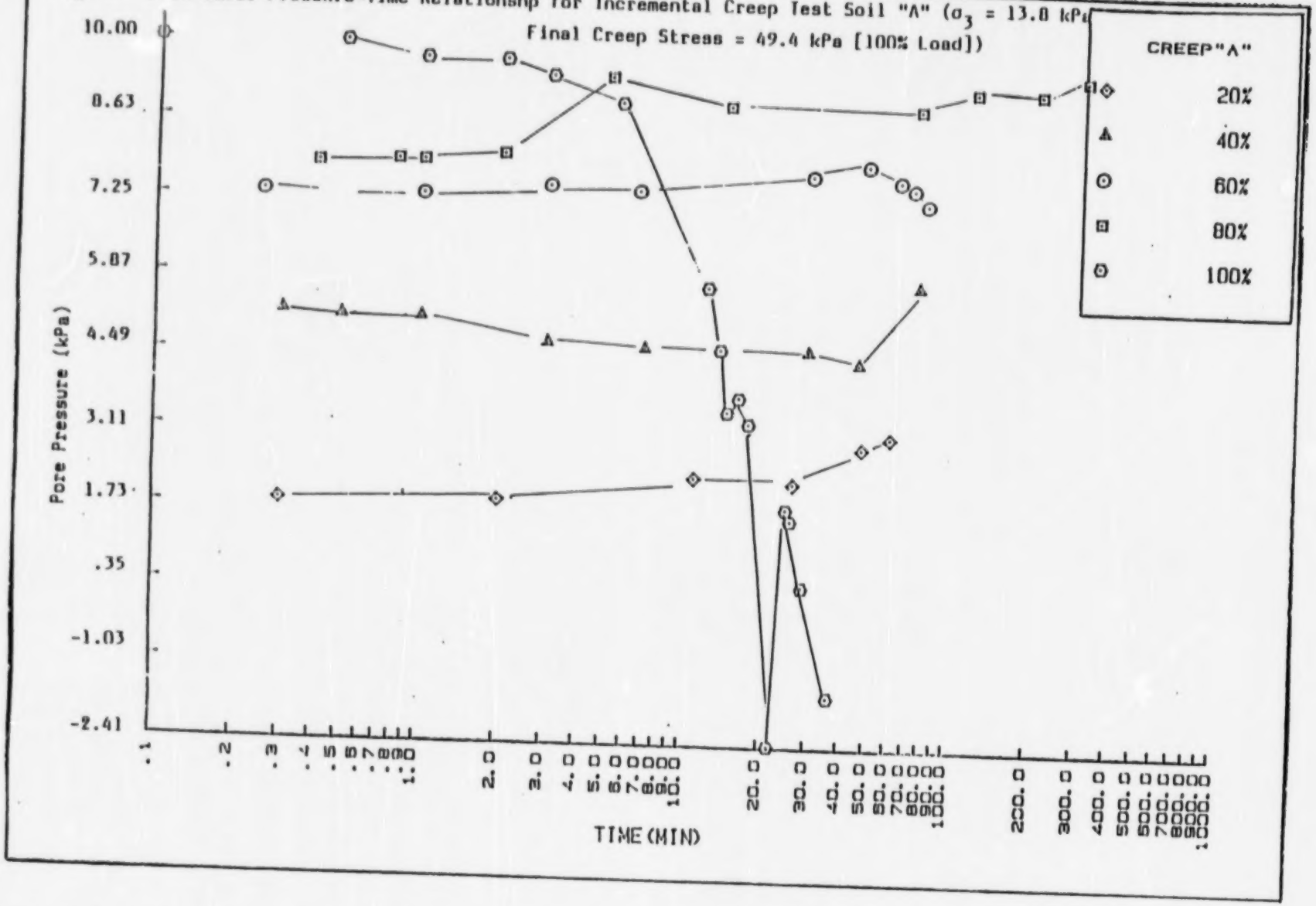


Fig. 87 Cumulative Strain-Time Relationship for Incremental Creep Test Soil "A" (σ_3 = 13.8 kPa
Final Creep Stress = 49.4 kPa [100% Load]).
CREEP "A"
20%
40%
60%
% CUM. STRAIN
.370
.333
.296
.259
.222
.185
.148
.111
.074
.037
0.000
TIME (MIN)

Fig. 88 Pore Water Pressure-Time Relationshp for Incremental Creep Test Soil "A" (σ_3 = 13.8 kPa
Final Creep Stress = 49.4 kPa [100% Load])



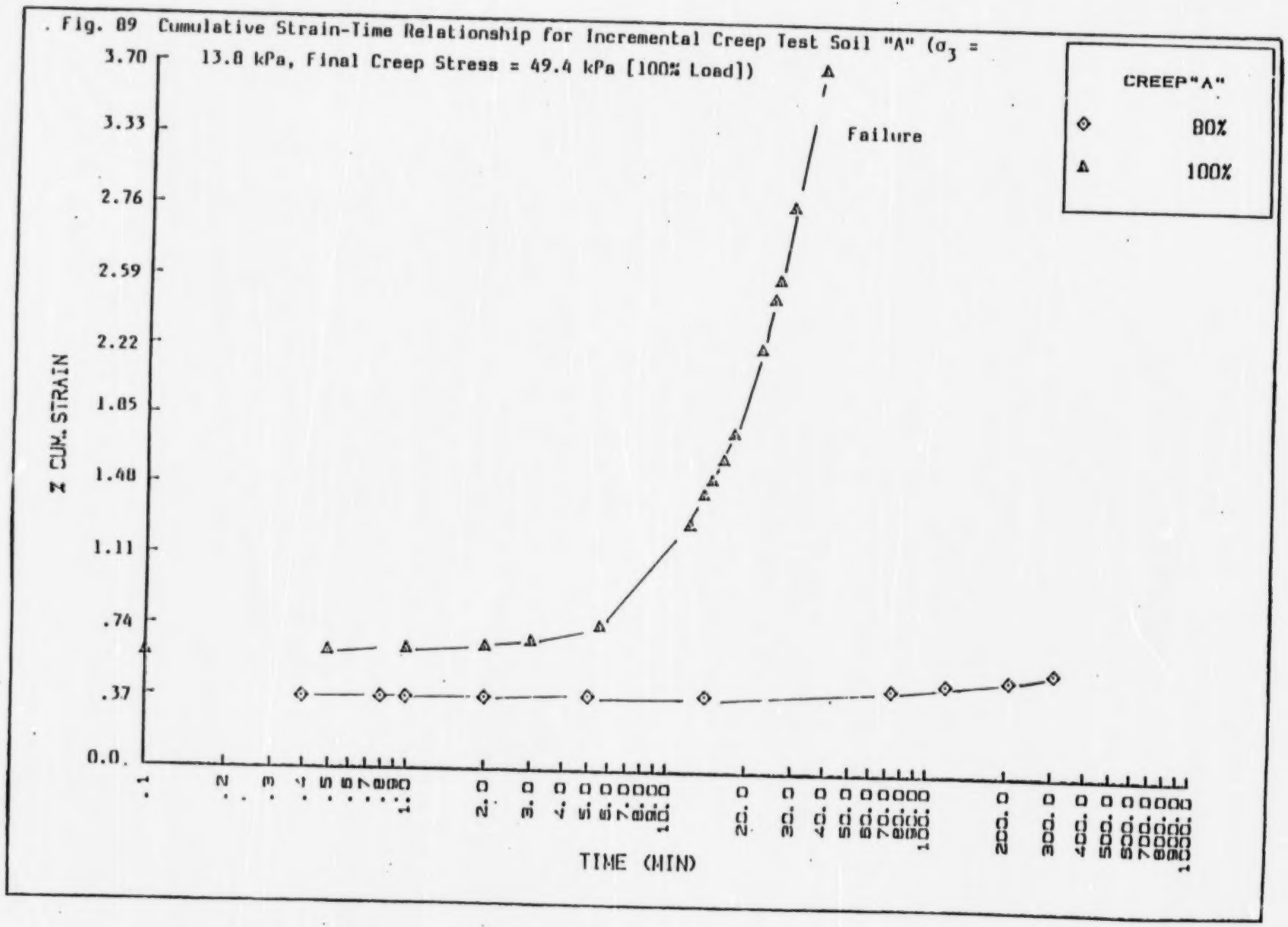
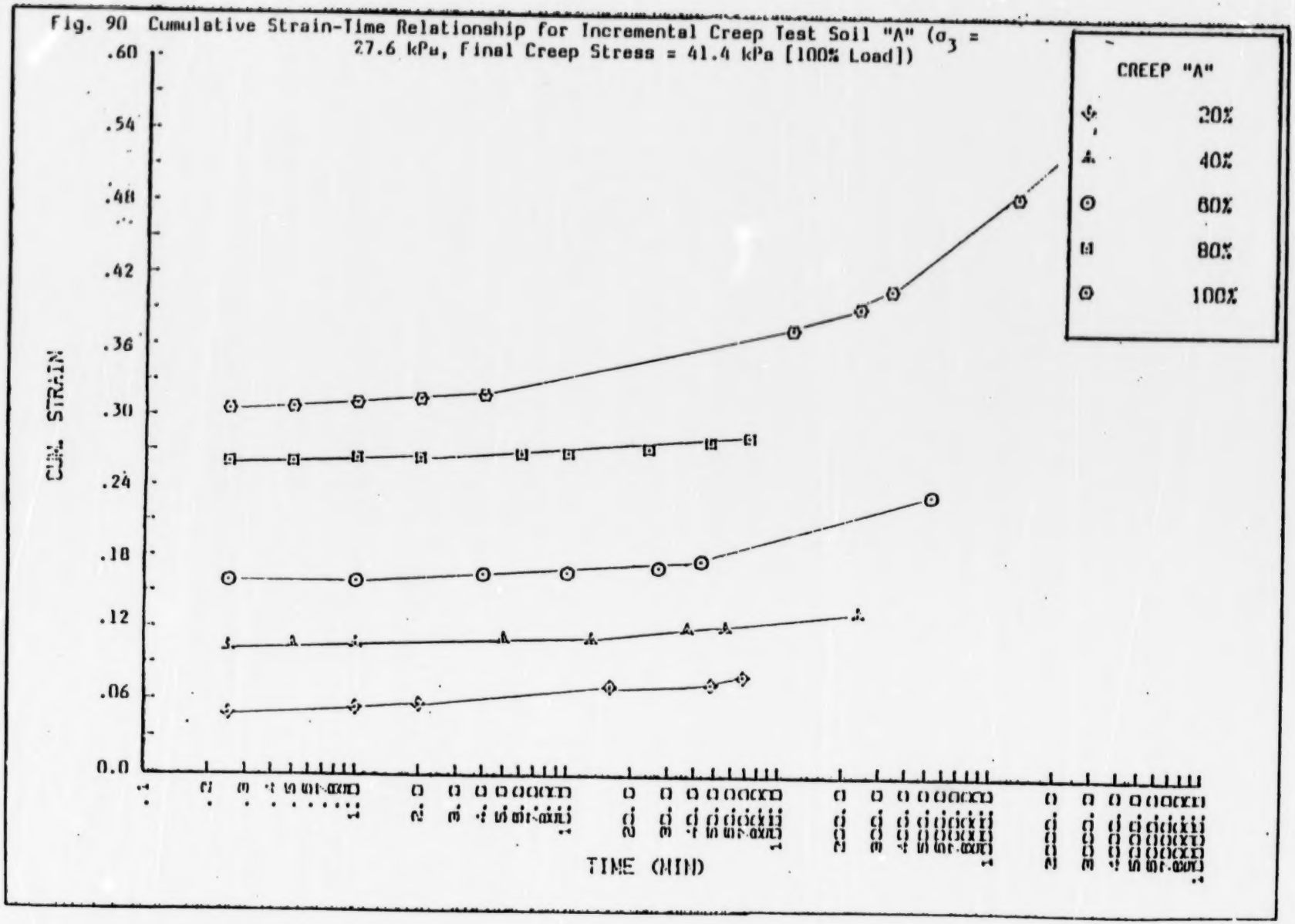


Fig. 89 Cumulative Strain-Time Relationship for Incremental Creep Test Soil "A" (σ_3 = 13.8 kPa, Final Creep Stress = 49.4 kPa [100% Load])
CREEP"A"
80%
100%
Failure
Z CUM. STRAIN
3.70
3.33
2.76
2.59
2.22
1.85
1.48
1.11
.74
.37
0.0
TIME (MIN)

Consolidation Pressure (kPa)	Back Pressure (kPa)	Final Creep Stress (kPa)	% of Final Creep Stress	% Cumulative Strain	Time (min)	Pore Water Pressure (kPa)	Remarks
27.6	275.8	41.4	20	.07	68	2.0	
			40	.14	237	5.9	
			60	.24	523	8.9	
			80	.30	72	11.7	
			100	.63	5515	13.5	no failure
27.6	275.8	46.7	20	.03	72	2.6	
			40	.18	60	5.9	
			60	.25	60	10.3	
			80	.35	60	13.1	
			100	.61	1012	17.4	failure
41.4	275.8	49.4	20	.07	65	3.6	
			40	1.02	56	9.6	
			60	1.14	60	14.0	
			80	1.25	60	18.2	
			100	2.15	7385	23.6	no failure

TABLE 11 Summary of Incremental Creep Tests on Soil "A" for σ_3 = 27.6 and 41.4 kPa

Fig. 90 Cumulative Strain-Time Relationship for Incremental Creep Test Soil "A" (σ_3 = 27.6 kPa, Final Creep Stress = 41.4 kPa [100% Load])



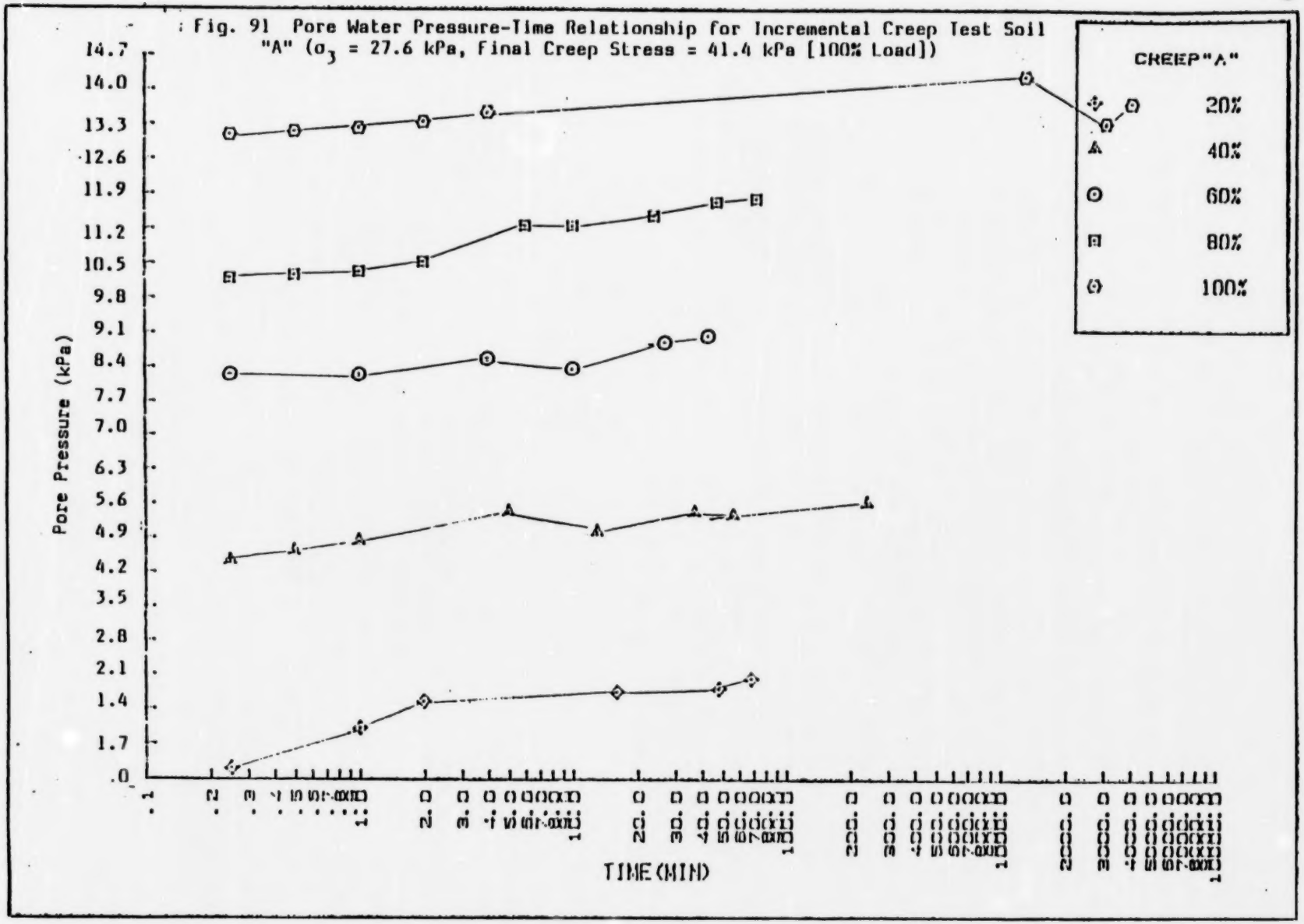


Fig. 91 Pore Water Pressure-Time Relationship for Incremental Creep Test Soil "A" (σ_3 = 27.6 kPa, Final Creep Stress = 41.4 kPa [100% Load])
CREEP"A"
20%
40%
60%
80%
100%
Pore Pressure (kPa)
14.7
14.0
13.3
12.6
11.9
11.2
10.5
9.8
9.1
8.4
7.7
7.0
6.3
5.6
4.9
4.2
3.5
2.8
2.1
1.4
1.7
.0
TIME(MIN)

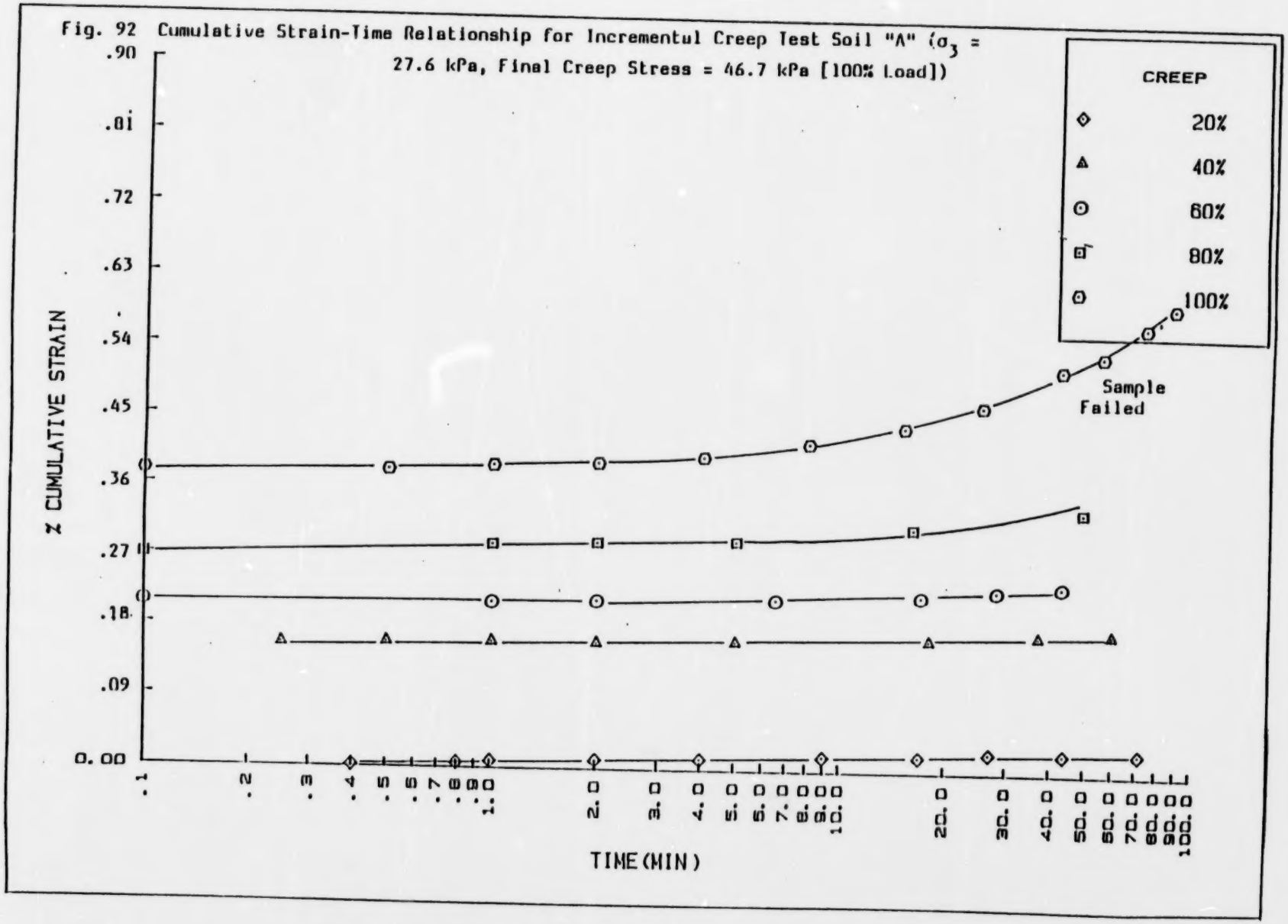


Fig. 92 Cumulative Strain-Time Relationship for Incrementul Creep Test Soil "A" (σ_3 = 27.6 kPa, Final Creep Stress = 46.7 kPa [100% Load])
CREEP
20%
40%
60%
80%
100%
Sample Failed
% CUMULATIVE STRAIN
.90
.81
.72
.63
.54
.45
.36
.27
.18
.09
0.00
.1
.2
.3
.4
.5
.6
.7
.8
.9
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10.0
20.0
30.0
40.0
50.0
60.0
70.0
80.0
90.0
100.
TIME(MIN)

Fig. 93 Pore Water Pressure-Time Relationship for Incremental Creep Test Soil "A"
(σ_3 = 27.6 kPa, Final Creep Stress = 46.7 kPa [100% Load])

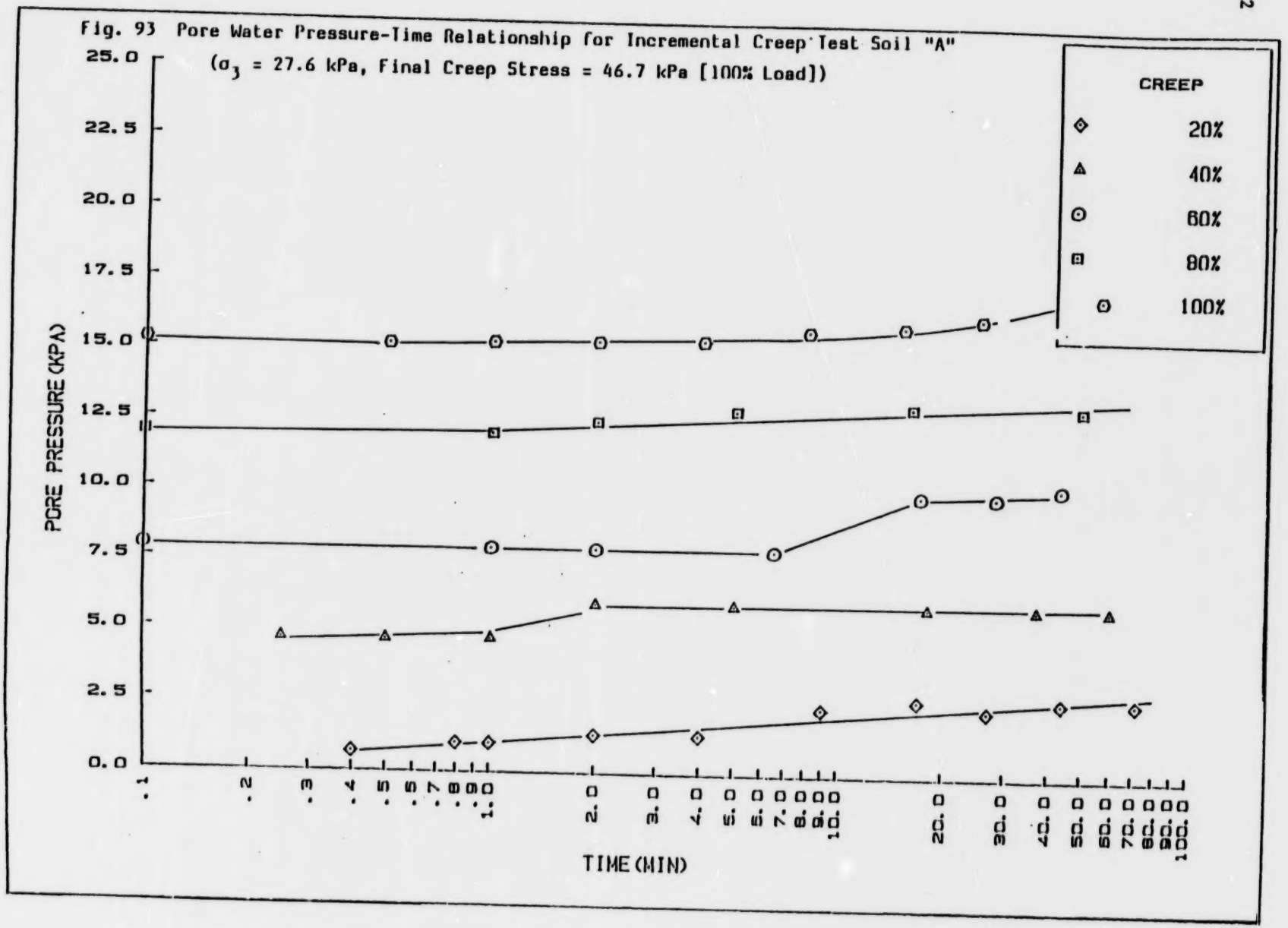


Fig. 94 Cumulative Strain-Time Relationship for Incremental Creep Test Soil "A" (σ_3 = 41.4 kPa, Final Creep Stress = 49.4 kPa [100% Load])

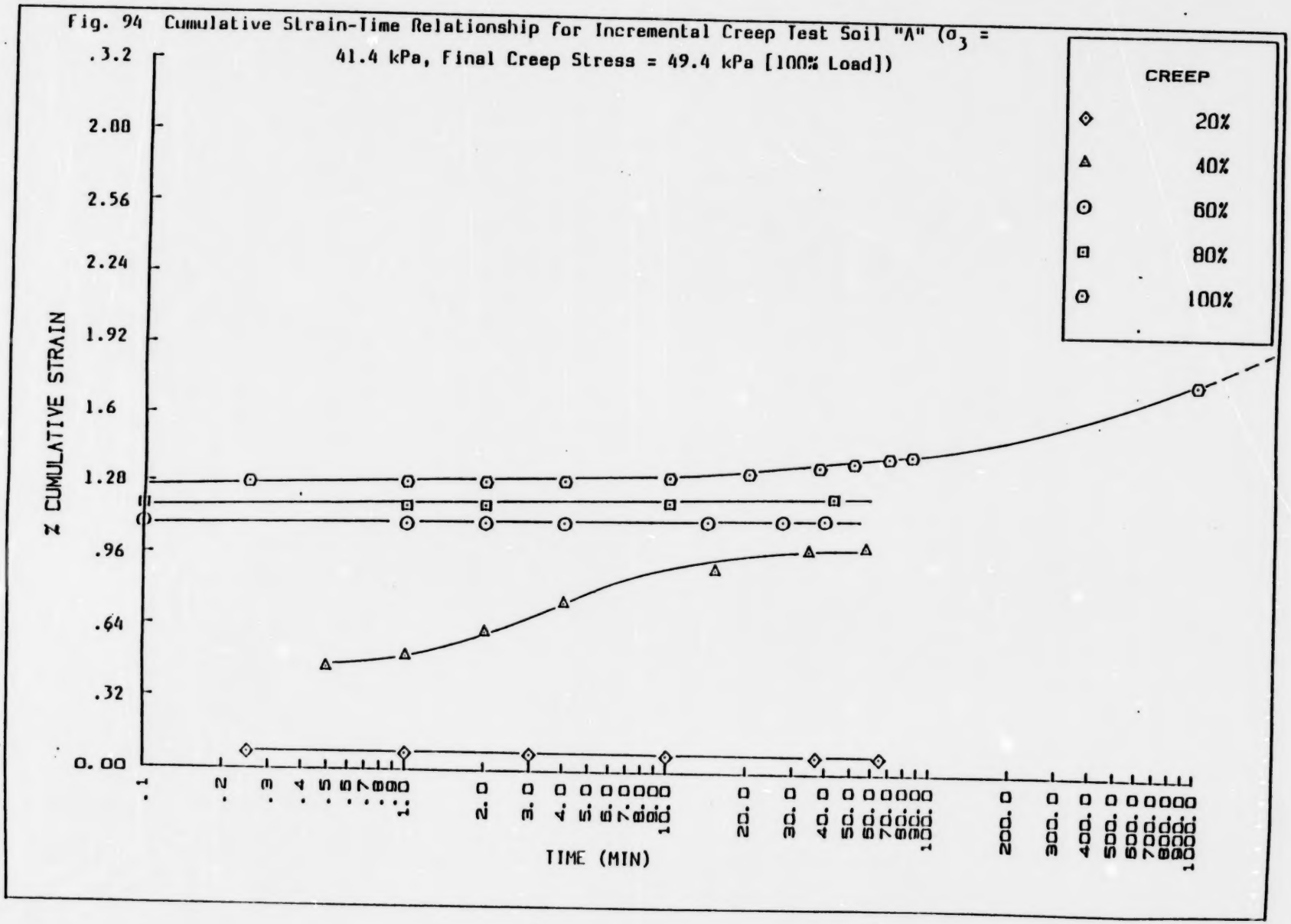
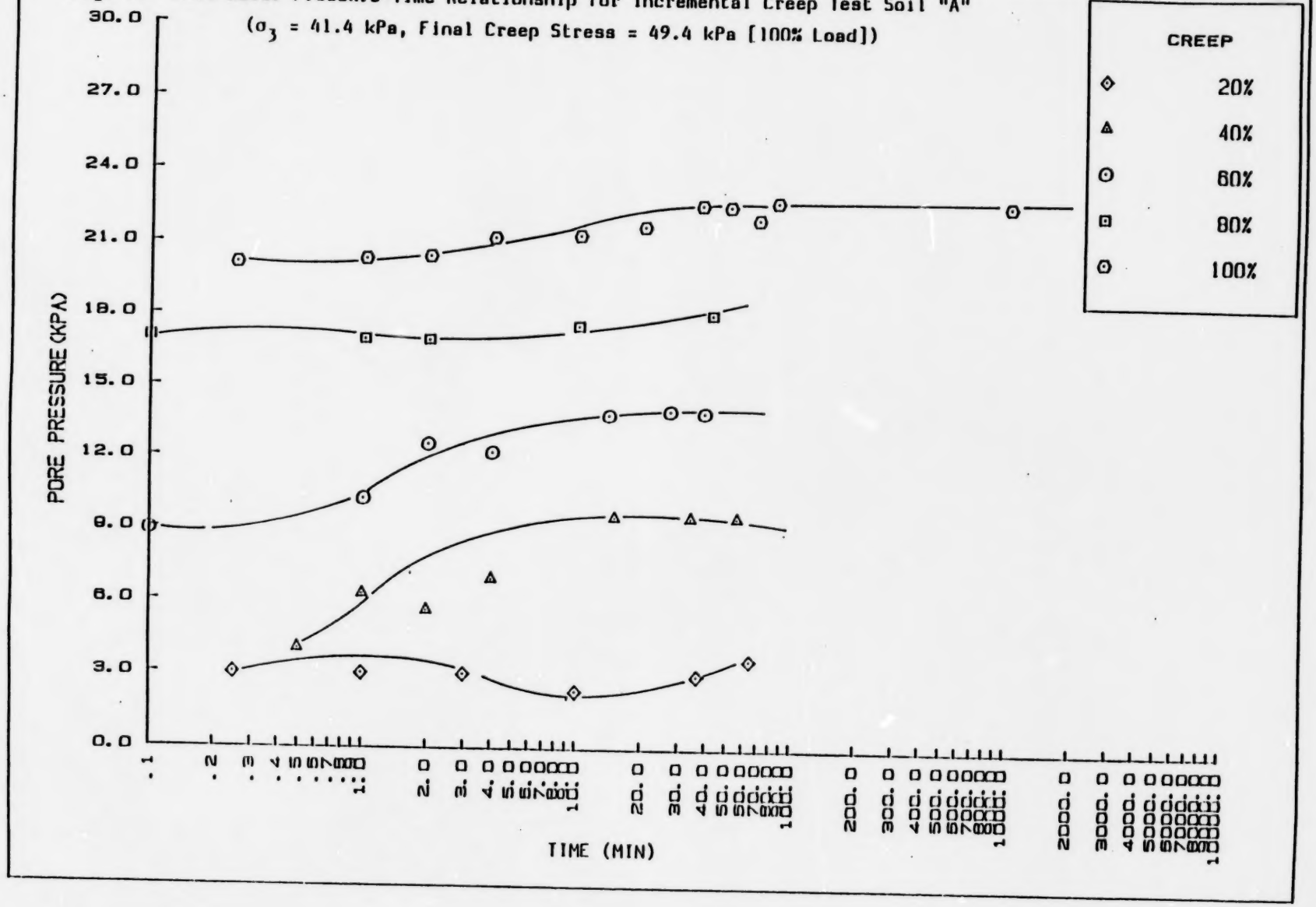


Fig. 95 Pore Water Pressure-Time Relationship for Incremental Creep Test Soil "A"
(σ_3 = 41.4 kPa, Final Creep Stress = 49.4 kPa [100% Load])



Consolidation Pressure (kPa)	Back Pressure (kPa)	Final Creep Stress (kPa)'	% of Final Creep Stress	% Cumulative Strain	Time (min)	Pore Water Pressure (kPa)	Remarks
13.8	275.8	38.4	20	.01	61	2.1	
			40	.16	67	0.7	
			60	.19	236	2.9	
			80	.21	57	5.0	
			100	.67	17170	N.A.	no failure
13.8	275.8	43.9	20	0.04	59	1.4	
			40	0.38	58	2.8	
			60	1.14	245	6.4	
			80	1.60	55	5.0	
			100	4.16	17200	-16.4	failure
27.6	275.8	41.1	20	.06	52	3.6	
			40	.19	75	2.7	
			60	.31	180	2.8	
			80	.40	63	3.5	
			100	1.23	9495	2.7	no failure
27.6	275.8	49.4	20	.21	72	2.7	
			40	.44	167	7.4	
			60	.72	77	11.0	
			80	1.11	62	15.3	
			100	6.0	6.0	15.9	failure
27.6	275.8	52.7	20	.06	60	N.A.	
			40	.20	198	1.4	
			60	.41	81	2.0	
			80	5.4	32	9.9	failure

TABLE 12 Summary of Incremental Creep Tests on Soil "B" σ_3 = 13.8 kPa

Consolidation Pressure (kPa)	Back Pressure (kPa)	Final Creep Stress (kPa)	% of Final Creep Stress	% Cumulative Strain	Time (min)	Pore Water Pressure (kPa)	Remarks
41.4	275.8	49.4	20	.11	67		
			40	.45	60	5.8	
			60	.53	66	9.0	
			80	.63	134	14.7	
			100	1.16	14473	31	no failure
41.4	275.8	52.7	20	.42	61	5.2	
			40	.58	61	11.7	
			60	1.36	112	22.1	
			80	6.46	5.0	24.1	failure

TABLE 12 Cont'd

Fig. 96 Cumulative Strain-Time Relationship for Incrmental Creep Test Soil "B"
(σ_3 = 13.8 kPa, Final Creep Stress = 38.4 kPa [100% Load])

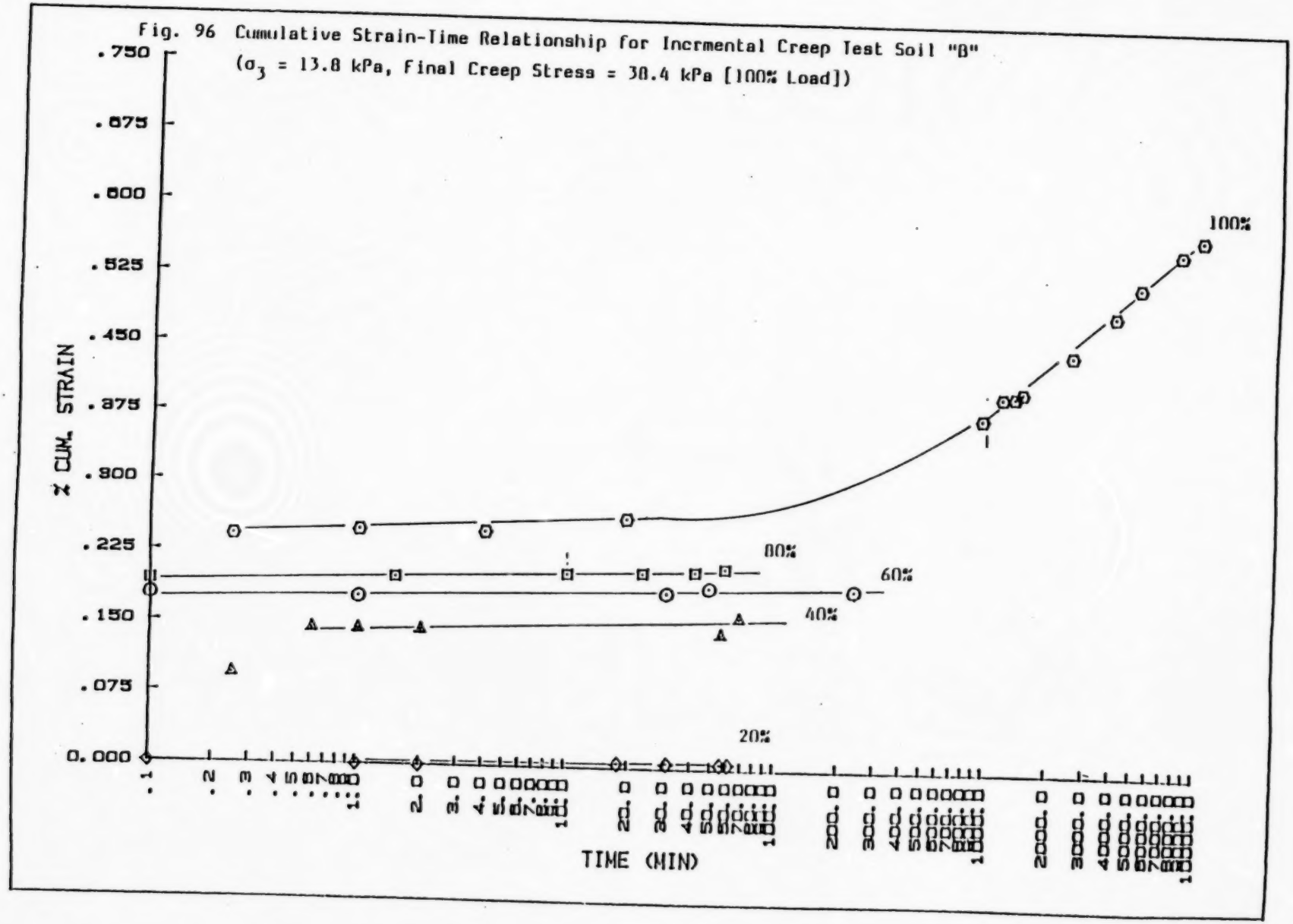


Fig. 97 Pore Water Pressure-Time Relationship for Incremental Creep Test Soil "B"
(σ_3 = 13.8 kPa, Final Creep Stress = 38.4 kPa [100% Load])

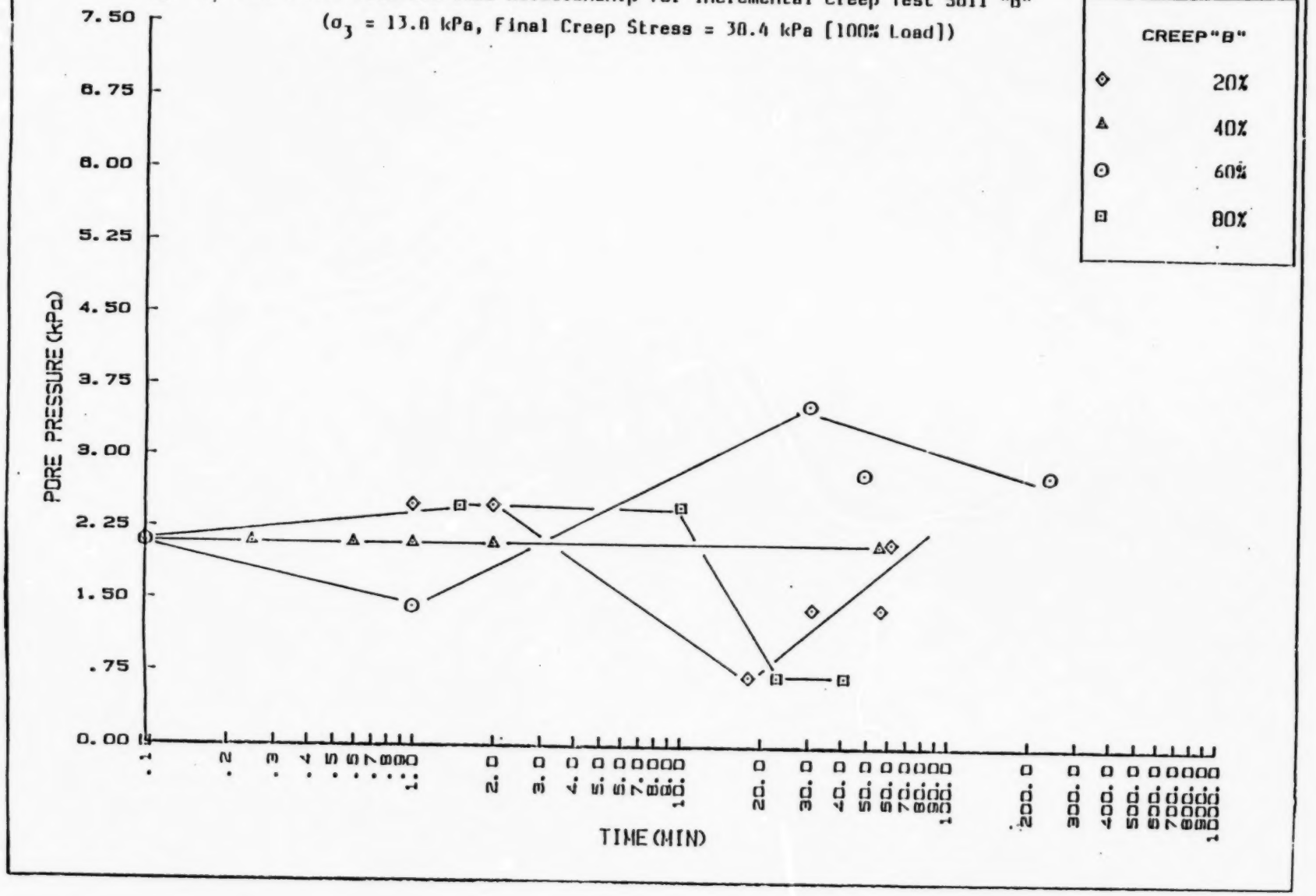


Fig. 98 Cumulative Strain-Time Relationship for Incremental Creep Test Soil "B"
(σ_3 = 13.8 kPa, Final Creep Stress = 43.9 kPa [100% Load])

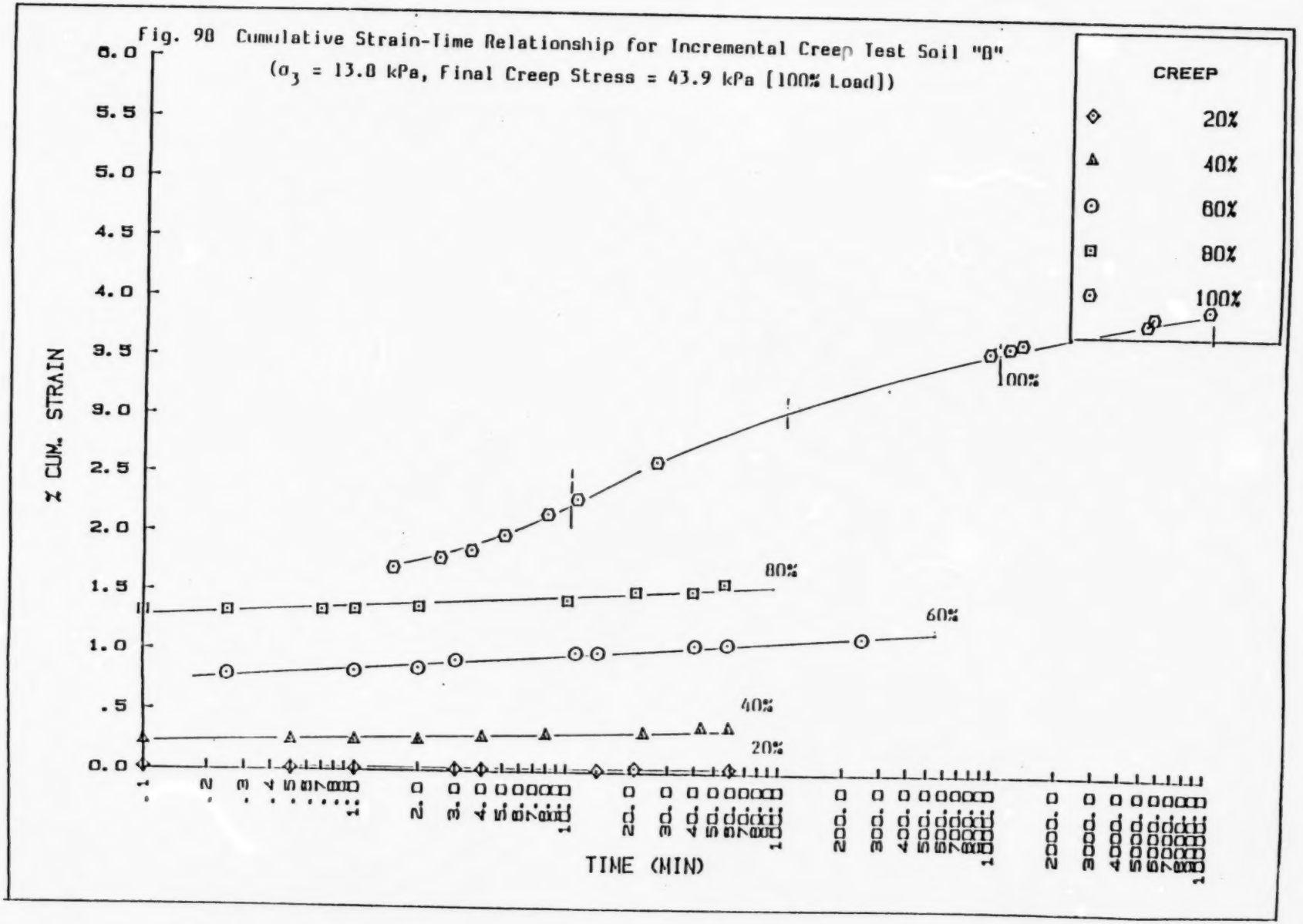


Fig. 99 Pore Water Pressure-Time Relationship for Incremental Creep Test Soil "B"
(σ_3 = 13.8 kPa, Final Creep Stress = 43.9 kPa [100% Load])

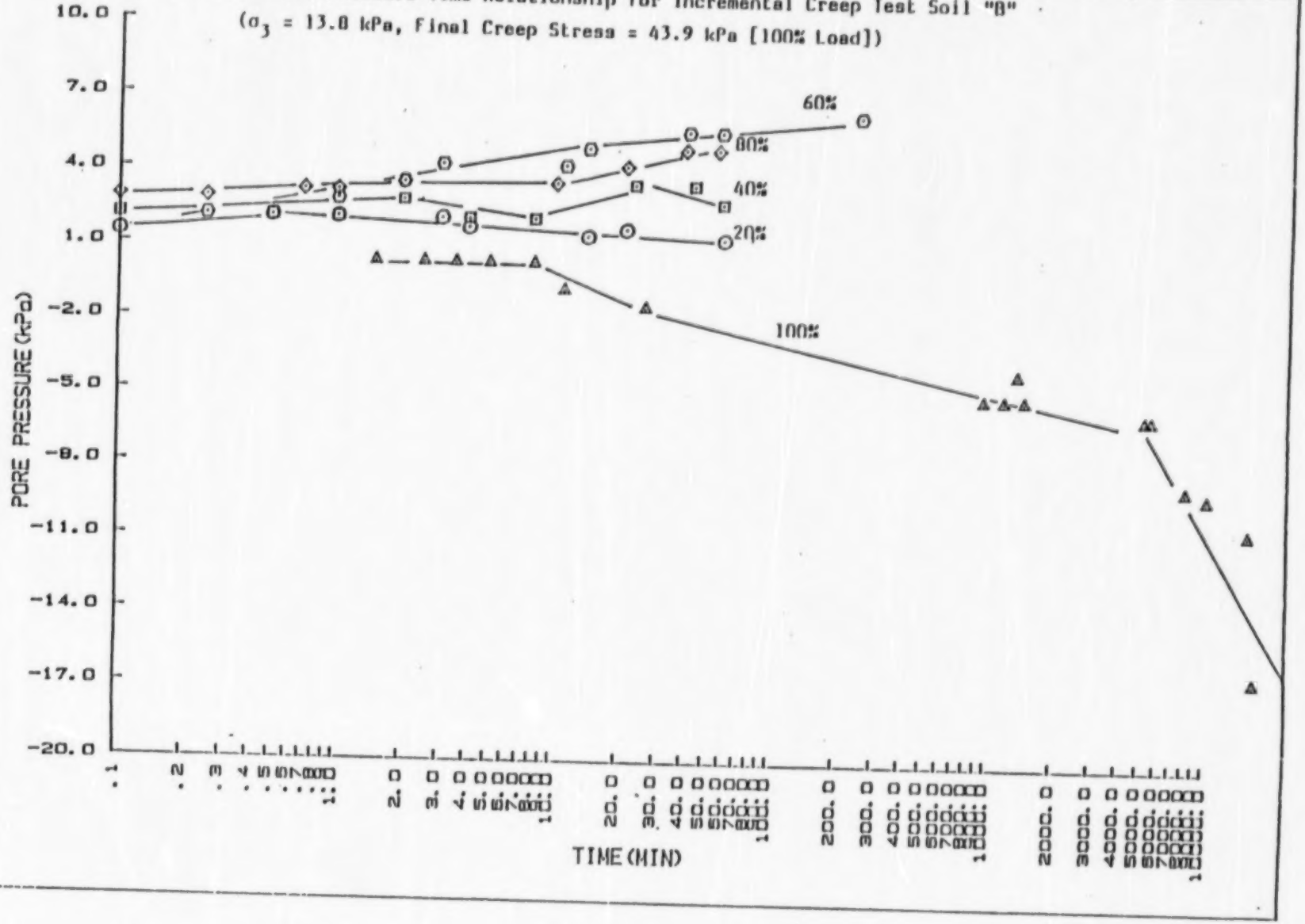


Fig. 100 Cumulative Strain-Time Relationship for Incremental Creep Tests Soil "B"
(σ_3 = 27.6 kPa, Final Creep Stress = 41.4 kPa [100% Load])

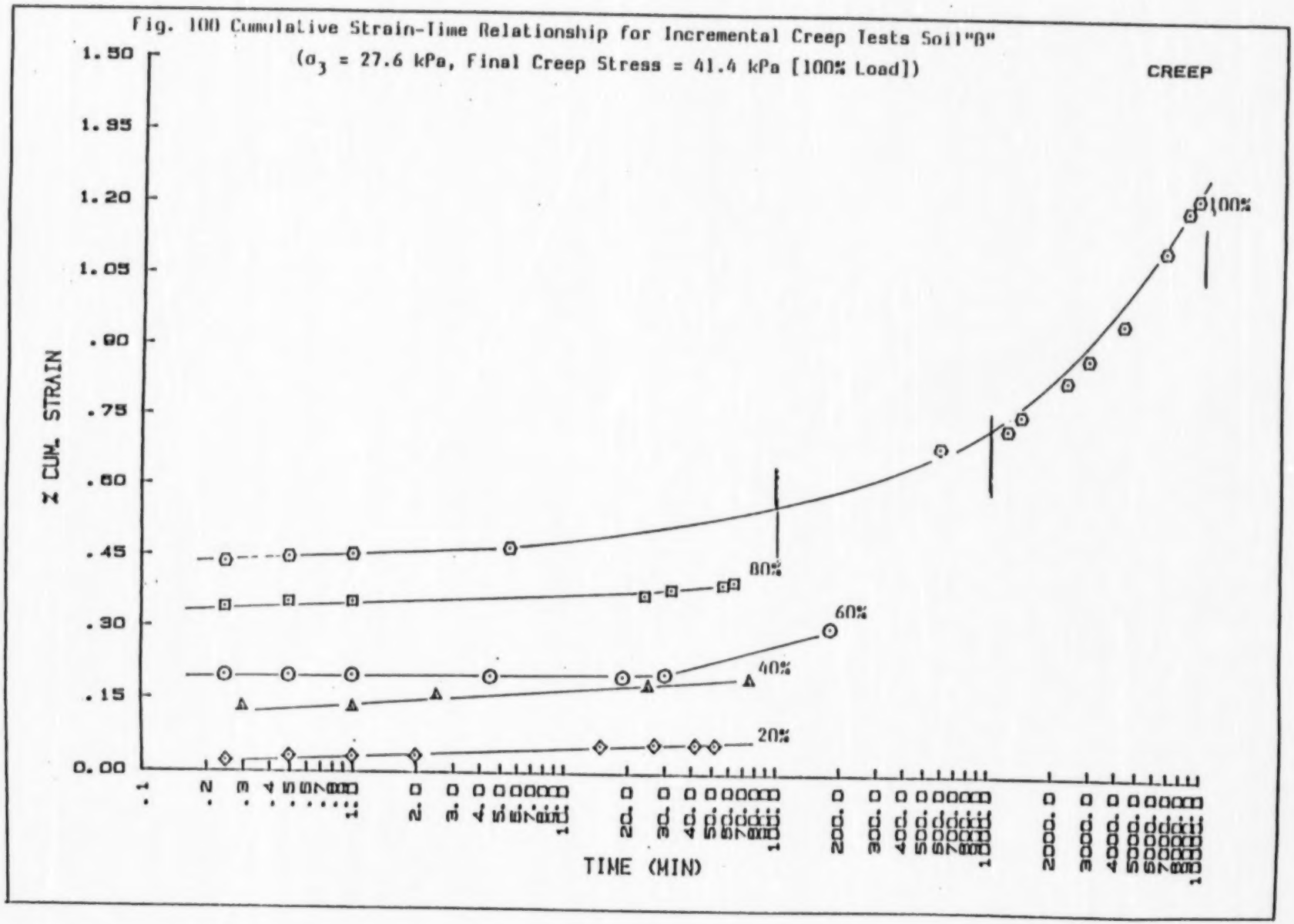
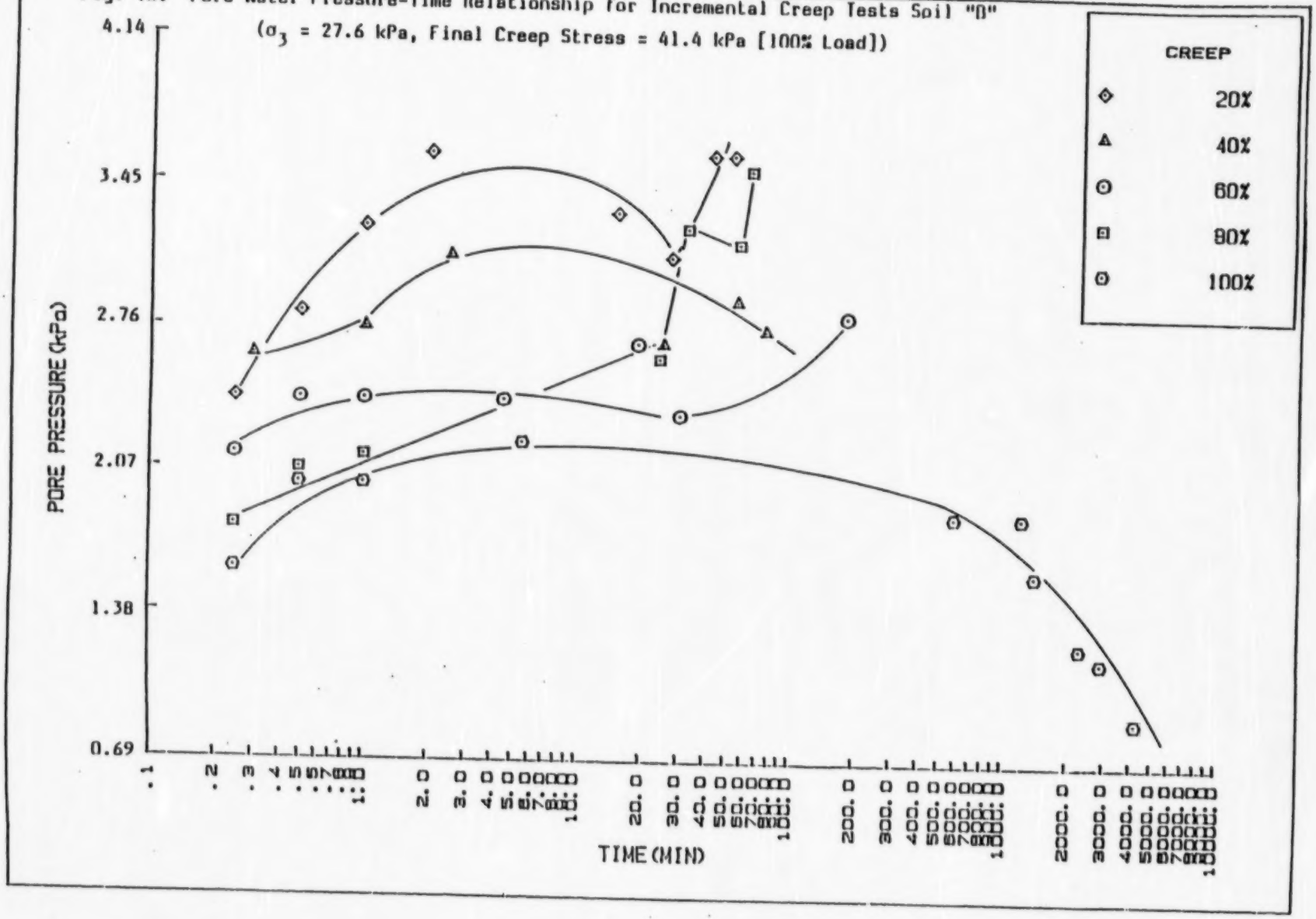


Fig. 101 Pore Water Pressure-Time Relationship for Incremental Creep Tests Soil "B"
(σ_3 = 27.6 kPa, Final Creep Stress = 41.4 kPa [100% Load])



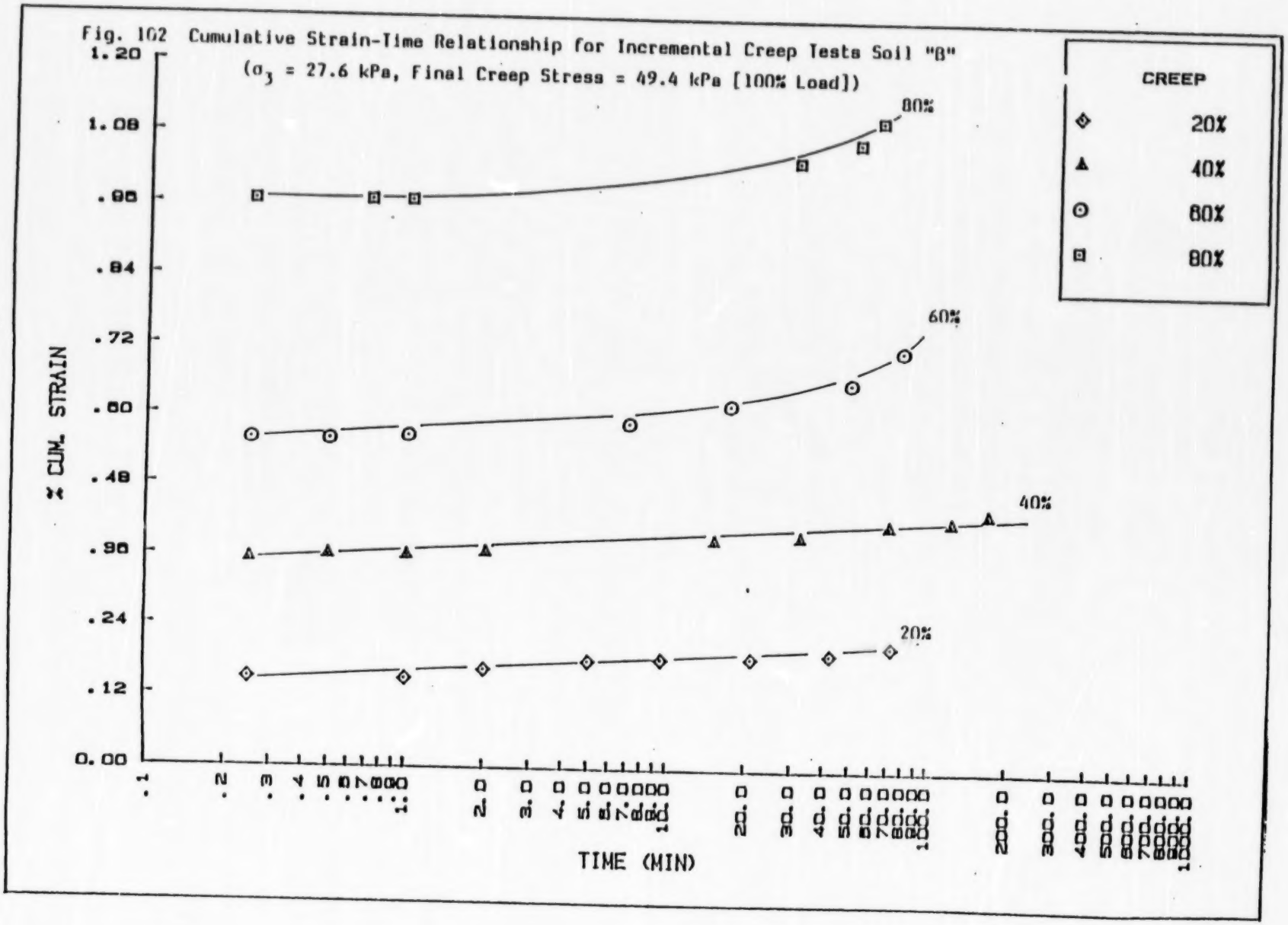
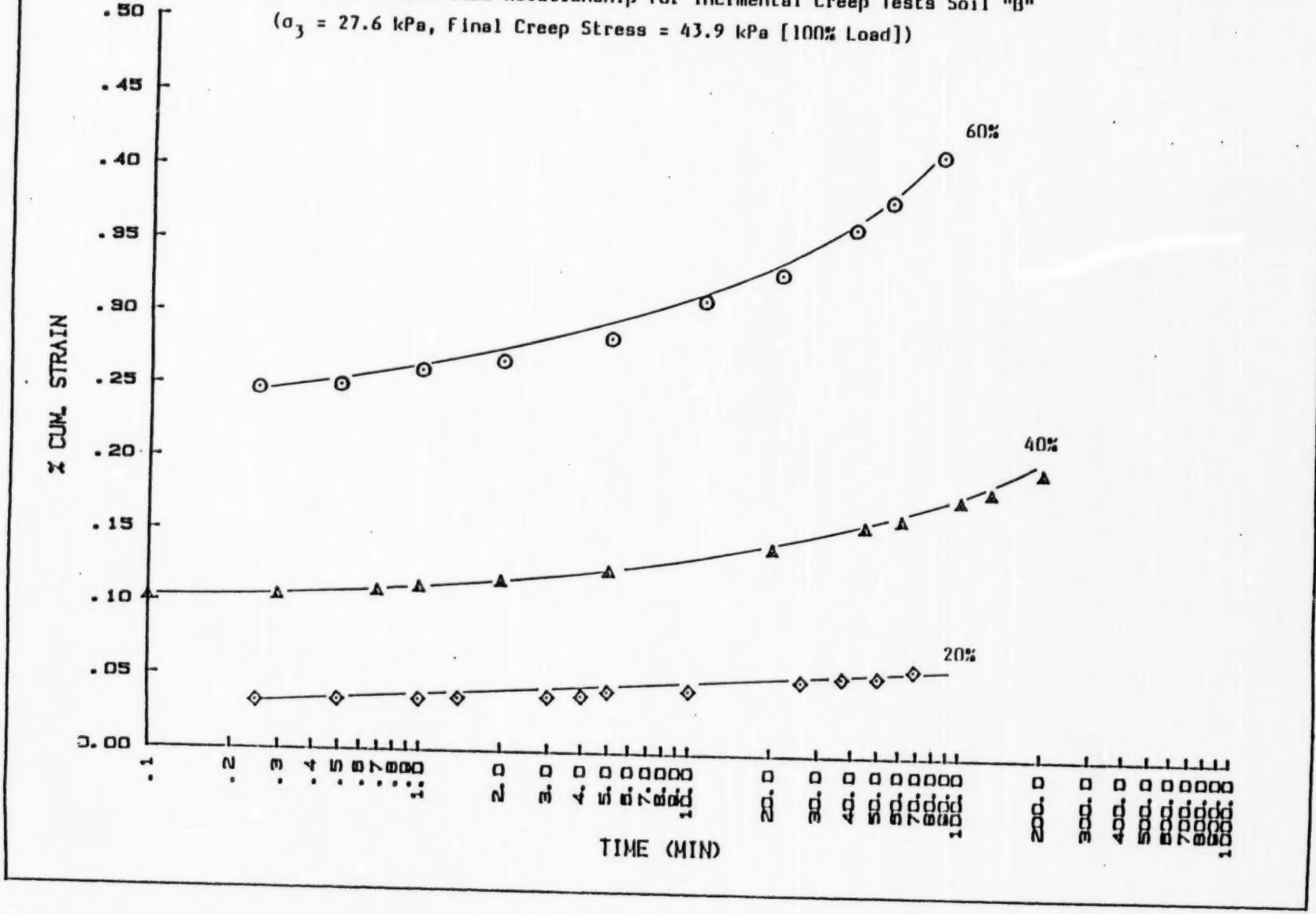


Fig. 102 Cumulative Strain-Time Relationship for Incremental Creep Tests Soil "B"
(σ_3 = 27.6 kPa, Final Creep Stress = 49.4 kPa [100% Load])
CREEP
20%
40%
60%
80%
% CUM. STRAIN
1.20
1.08
.96
.84
.72
.60
.48
.36
.24
.12
0.00
TIME (MIN)

Fig. 103 Cumulative Strain-Time Relationship for Incremental Creep Tests Soil "B"
(σ_3 = 27.6 kPa, Final Creep Stress = 43.9 kPa [100% Load])



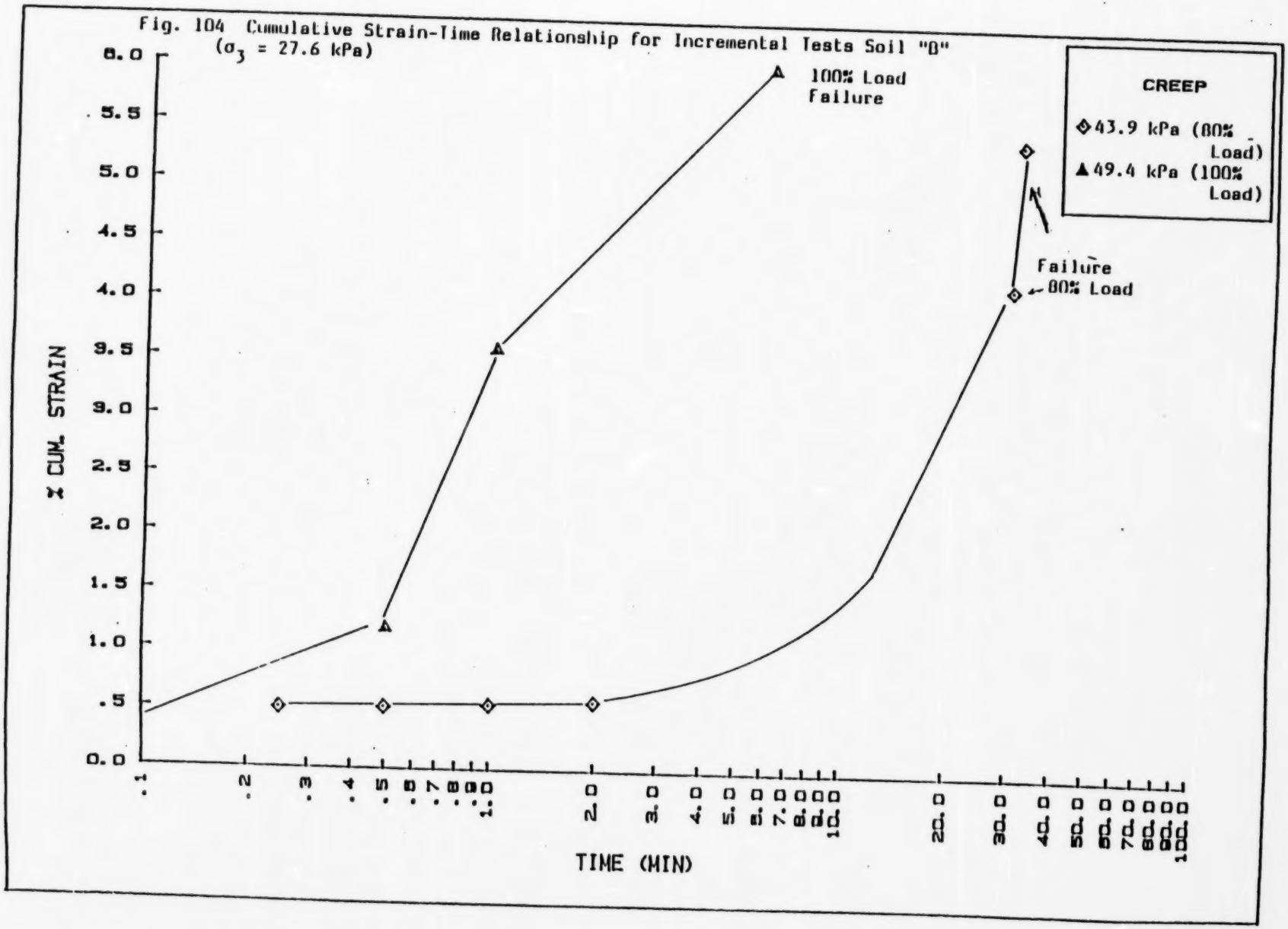


Fig. 104 Cumulative Strain-Time Relationship for Incremental Tests Soil "B"
(σ_3 = 27.6 kPa)
100% Load
Failure
CREEP
43.9 kPa (80% Load)
49.4 kPa (100% Load)
Failure
80% Load
% CUM. STRAIN
TIME (MIN)

Fig. 105 Pore Water Pressure-Time Relationship for Incremental Creep Tests Soil "B"
(σ_3 = 27.6 kPa, Final Creep Stress = 49.4 kPa [100% Load])

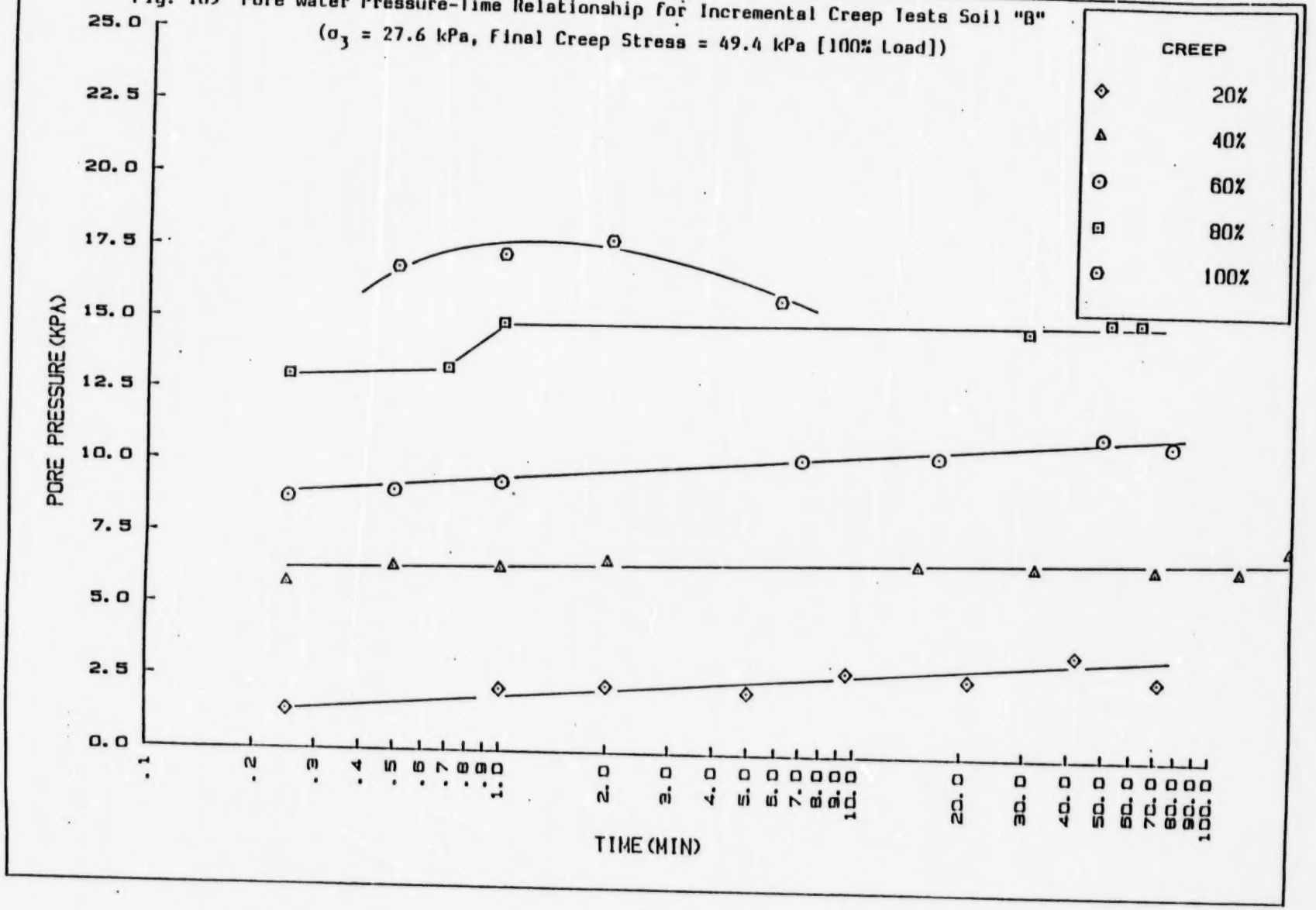


Fig. 106 Pore Pressure-Time Relationship for Incremental Creep Tests Soil "B"
(σ_3 = 27.6 kPa, Final Creep Stress = 43.9 kPa [80% Load])

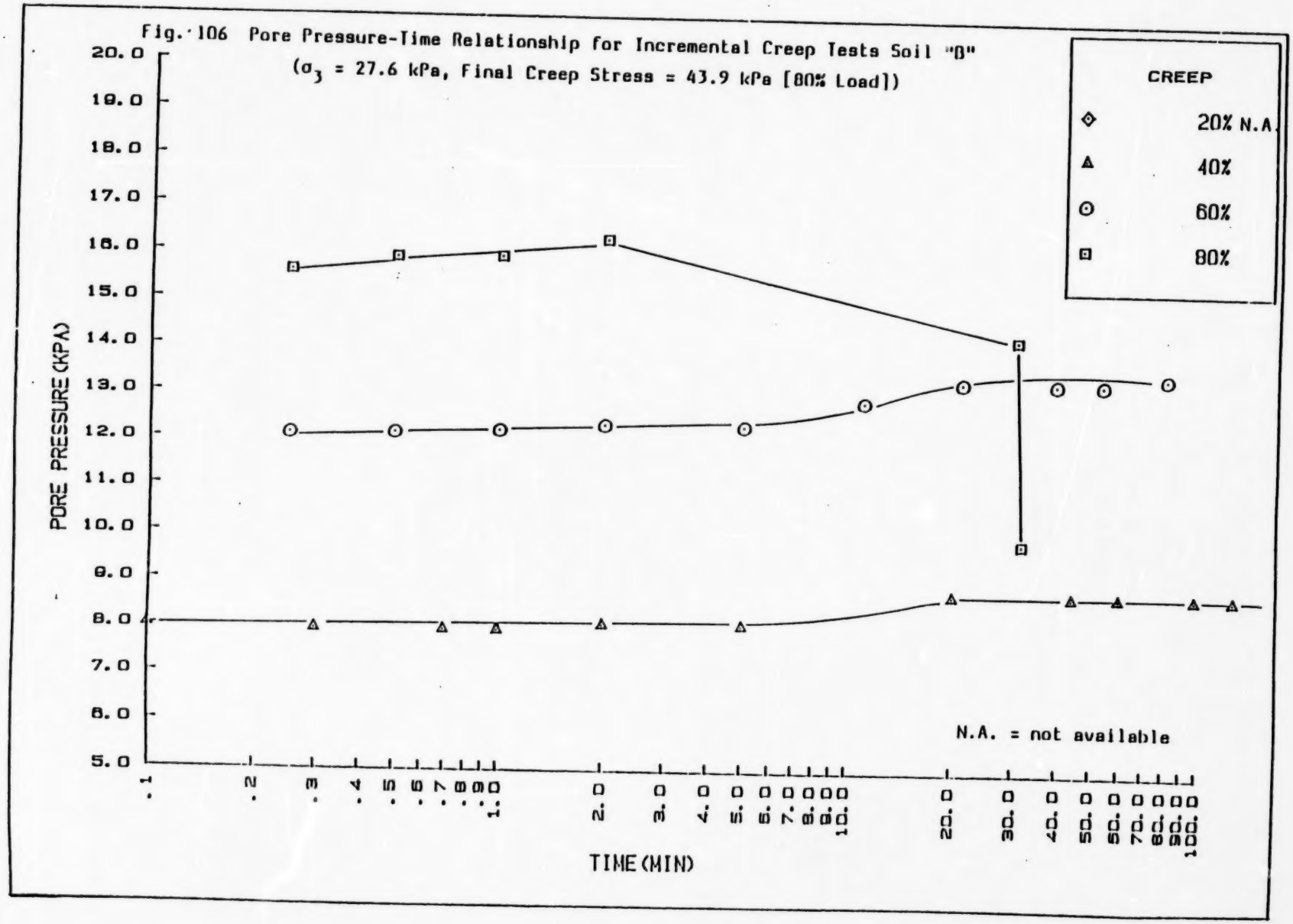


Fig. 107 Cumulative Strain-Time Relationship for Incrmeental Creep Tests Soil "B"
(σ_3 = 41.4 kPa, Final Creep Stress = 49.4 kPa [100% Load])

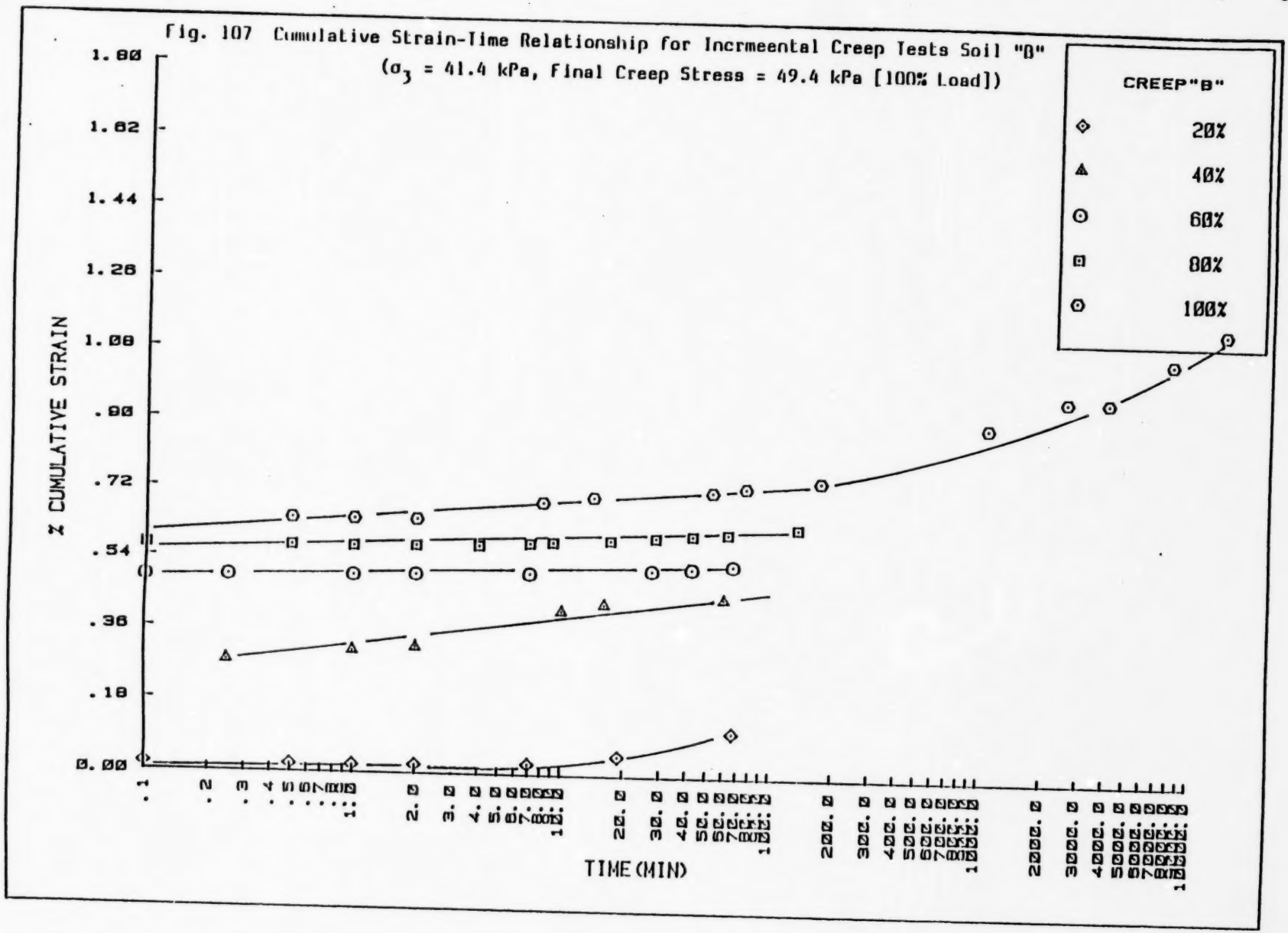


Fig. 108 Pore Water Pressure-Time Relationship for Incremental Creep Test Soil "B"
(σ_3 = 41.4 kPa, Final Creep Stress = 49.4 kPa [100% Load])

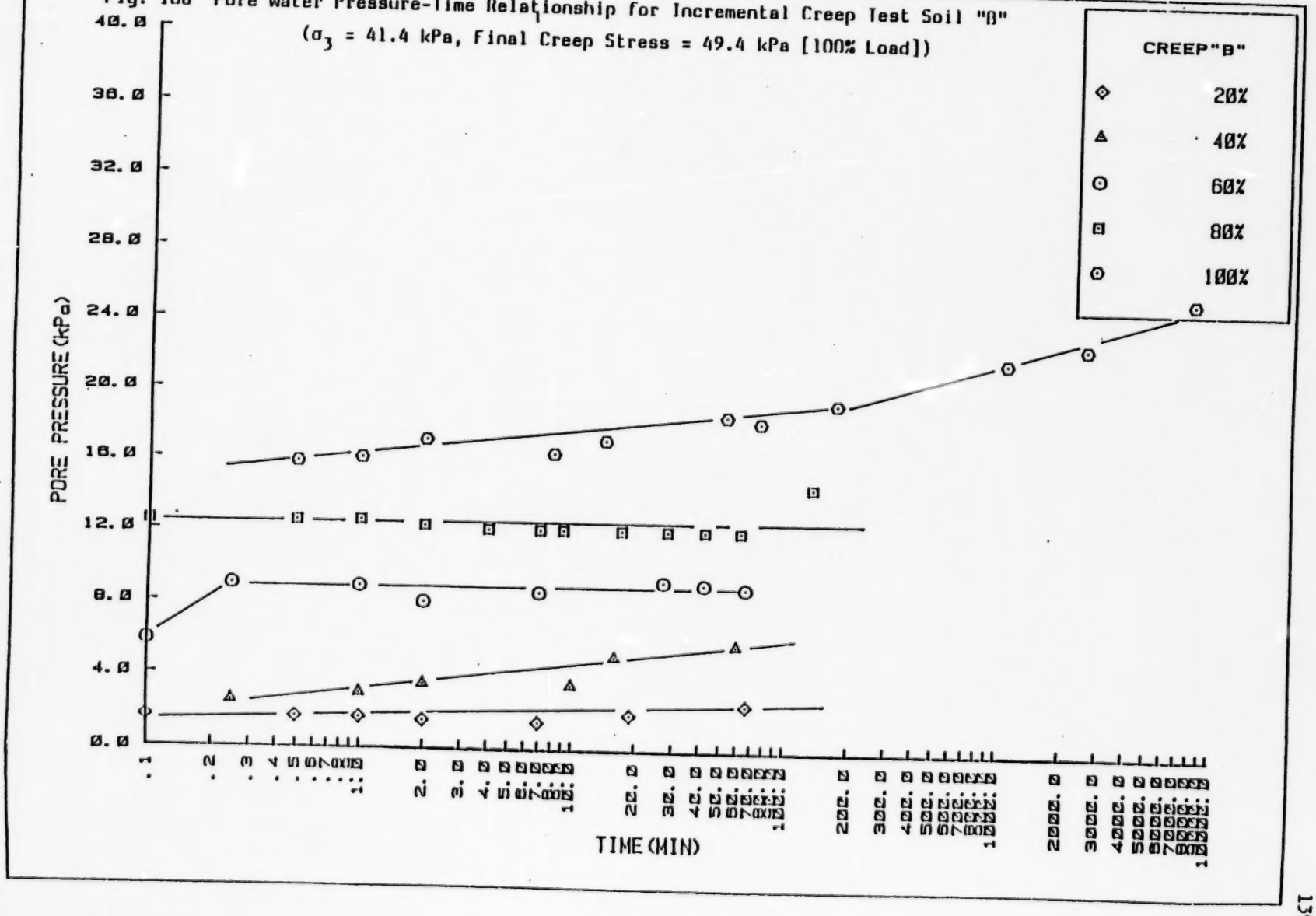
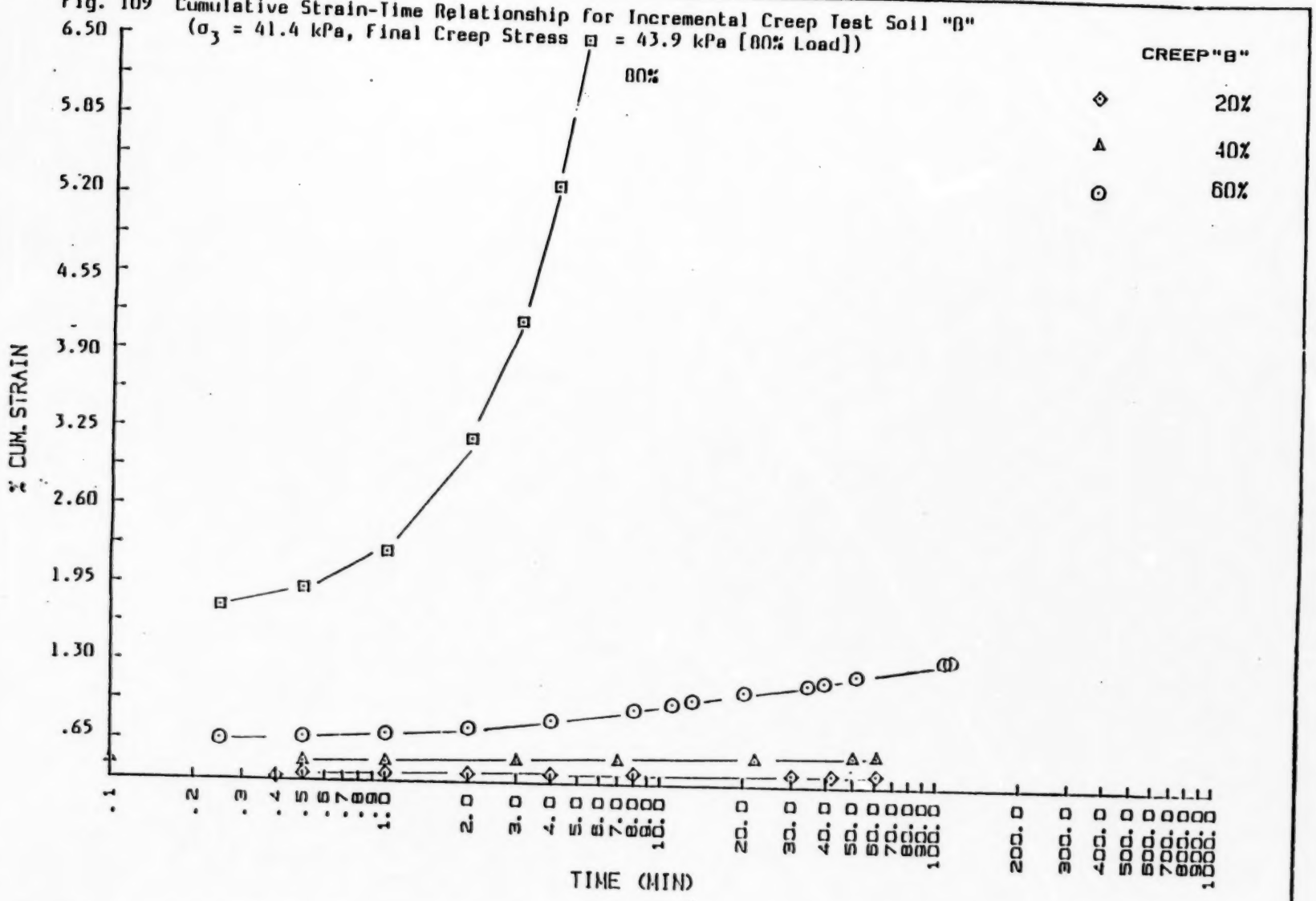


Fig. 109 Cumulative Strain-Time Relationship for Incremental Creep Test Soil "B"
(σ_3 = 41.4 kPa, Final Creep Stress ⊡ = 43.9 kPa [80% Load])



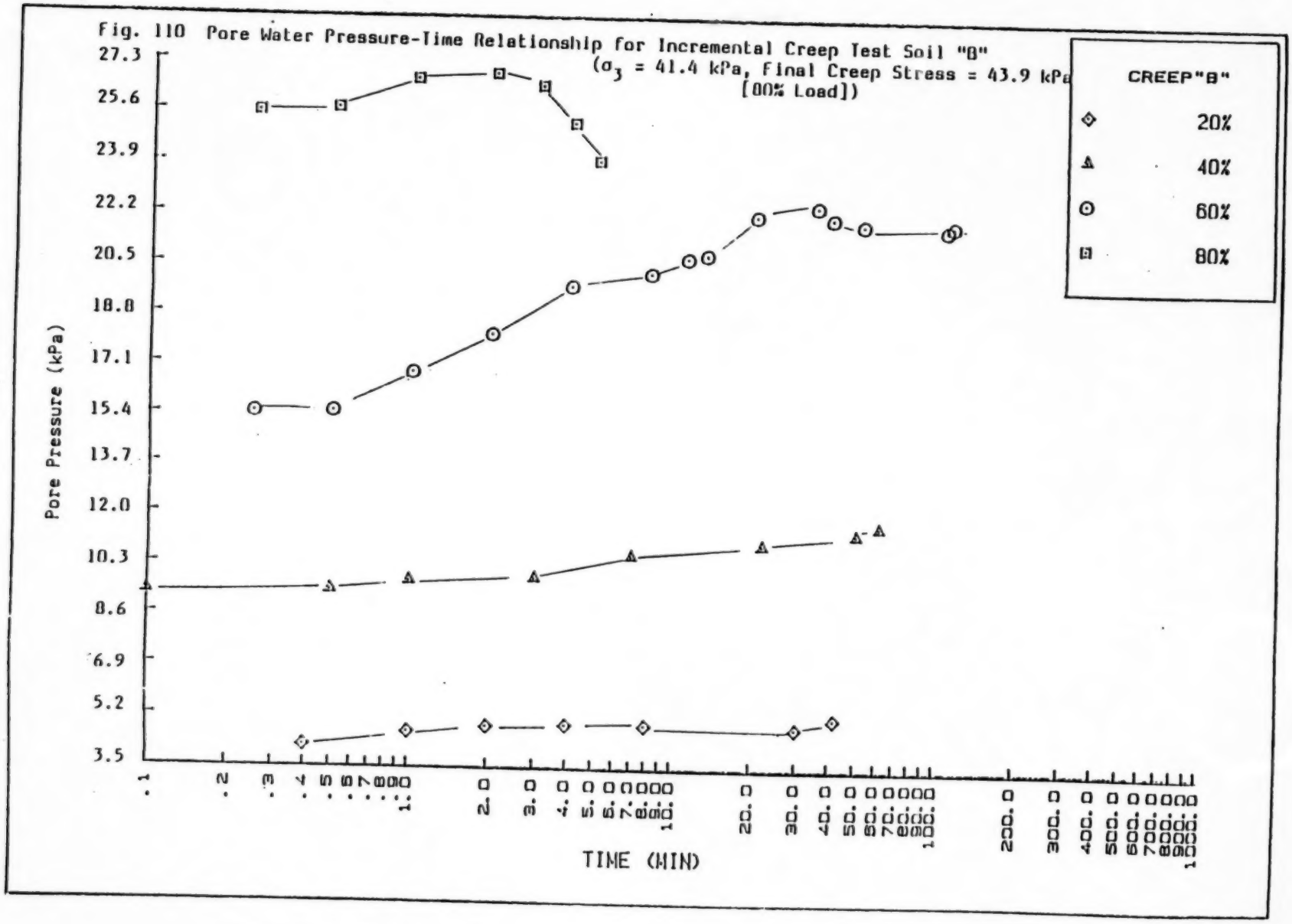


Fig. 110 Pore Water Pressure-Time Relationship for Incremental Creep Test Soil "B"
(σ_3 = 41.4 kPa, Final Creep Stress = 43.9 kPa
[80% Load])
CREEP"B"
20%
40%
60%
80%
Pore Pressure (kPa)
27.3
25.6
23.9
22.2
20.5
18.8
17.1
15.4
13.7
12.0
10.3
8.6
6.9
5.2
3.5
TIME (MIN)

Consolidation Pressure (kPa)	Back Pressure (kPa)	Final Creep Stress (kPa)	% of Final Creep Stress	% Cumulative Strain	Time (min)	Pore Water Pressure (kPa)	Remarks
13.8	275.8	19.3	20	-	60	-	
			40	-	118	-	
			60	.06	44	N.A.	
			80	.12	88	N.A.	
			100	.34	2581	N.A.	no failure
13.8	275.8	22.1	20	.10	60	2.0	
			40	.11	61	2.7	
			60	.18	68		
			80	.26	90		
			100	.53	5813	2.0	no failure
27.6	275.8	27.6	20	nil	-	-	
			40		-	-	
			60		-	-	
			80	nil	-	-	
			100	.21	7592	4.7	no failure
27.6	275.8	33.1	20	nil	-	-	
			40		-	-	
			60		-	-	
			80	.05	62	5.9	
			100	.47	7263	5.9	no failure
27.6	275.8	34.5	20	nil	-	-	
			40	nil	-	-	
			60	.09	1133	5.6	
			80	.12	70	7.9	
			100	.36	10227	6.6	no failure

TABLE 13 Summary of Incremental Creep Tests on Joliette Soil

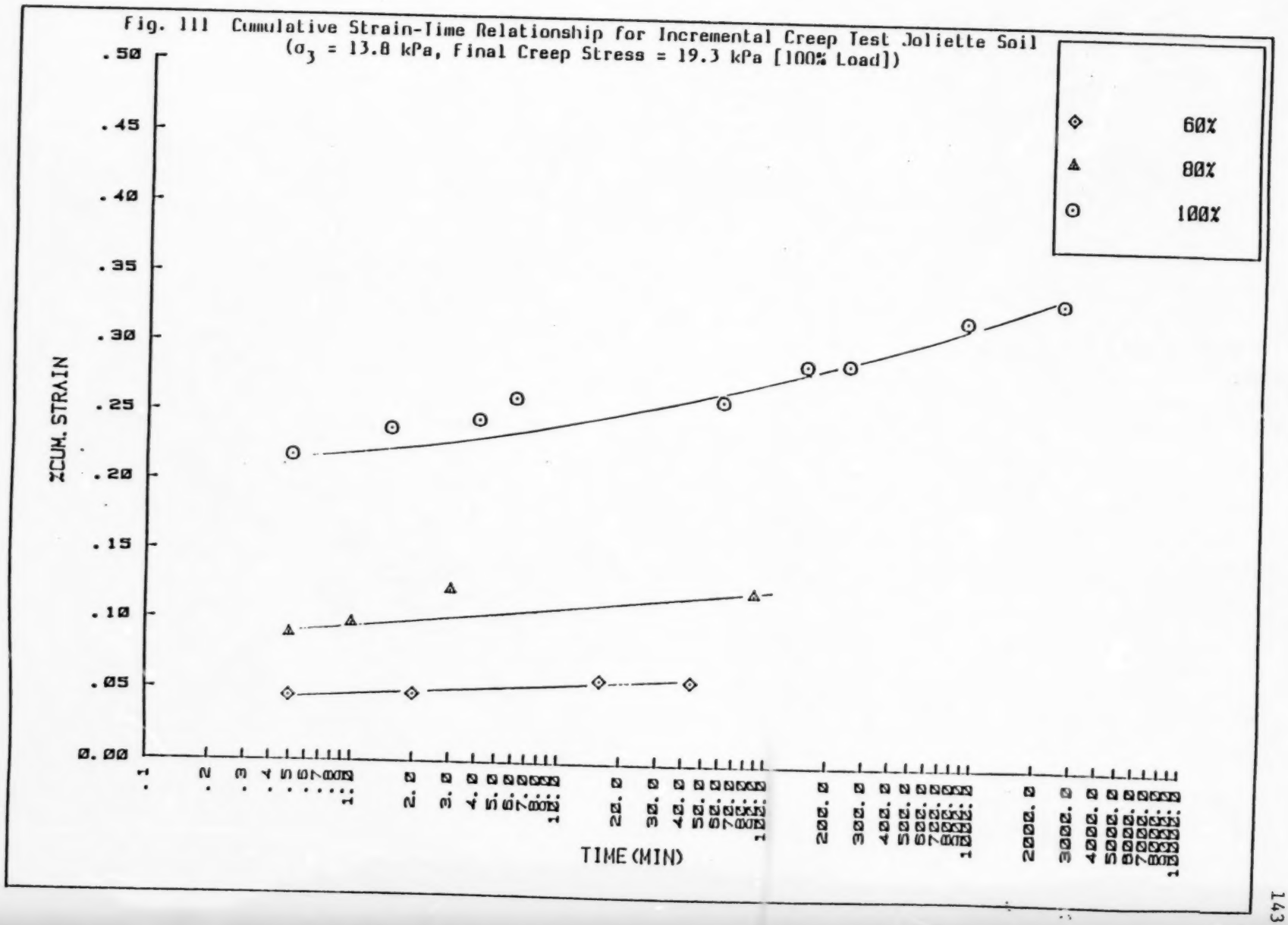


Fig. 111 Cumulative Strain-Time Relationship for Incremental Creep Test Joliette Soil
(σ_3 = 13.8 kPa, Final Creep Stress = 19.3 kPa [100% Load])
60%
80%
100%
%CUM. STRAIN
.50
.45
.40
.35
.30
.25
.20
.15
.10
.05
0.00
TIME (MIN)

Fig. 112 Cumulative Strain-Time Relationship for Incremental Creep Test Joliette Soil
(σ_3 = 13.8 kPa, Final Creep Stress = 22.1 kPa [100% Load])

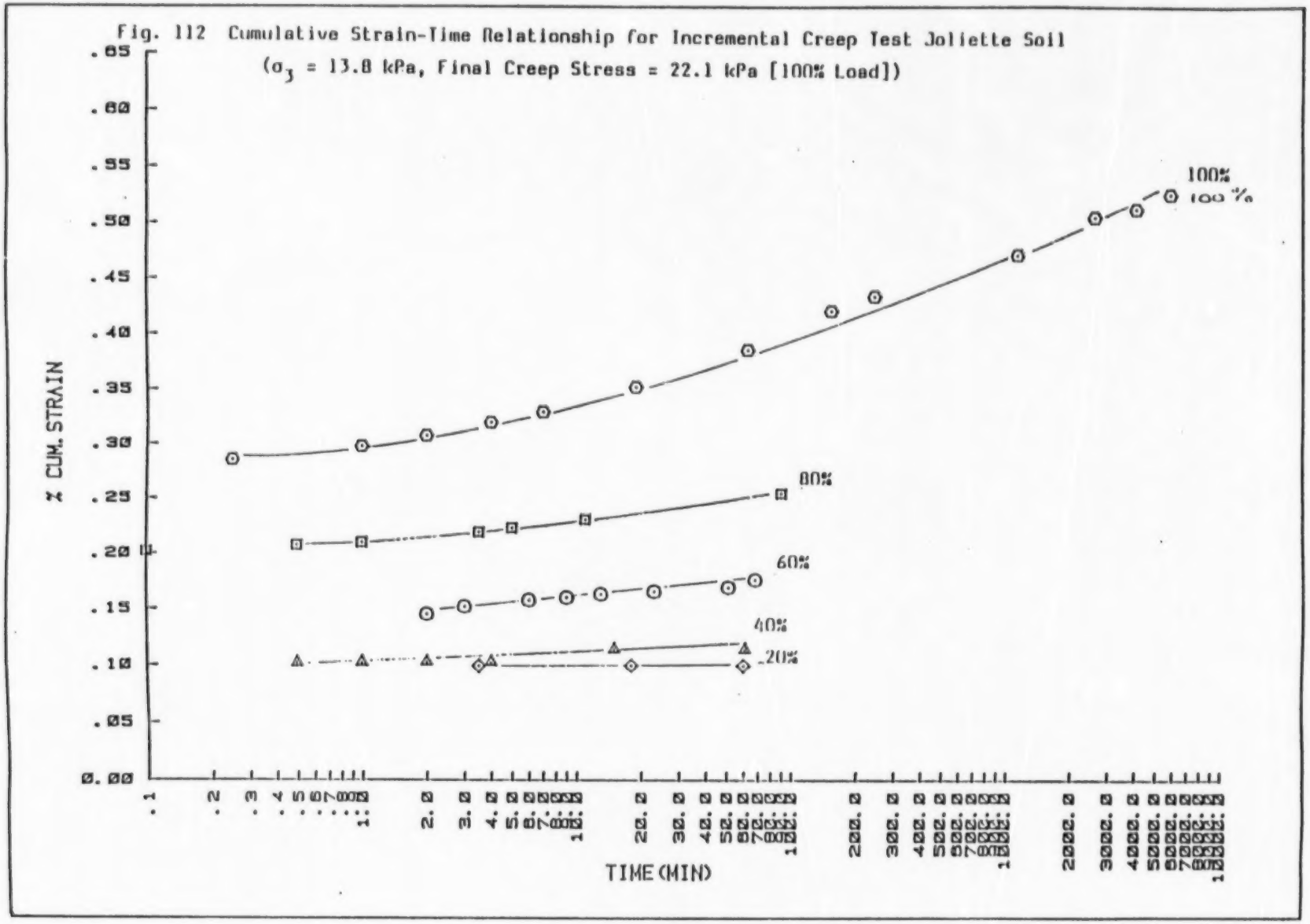


Fig. 113 Pore Water Pressure-Time Relationship for Incremental Creep Test Joliette Soil
(σ_3 = 13.8 kPa, Final Creep Stress = 22.1 kPa [100% Load])

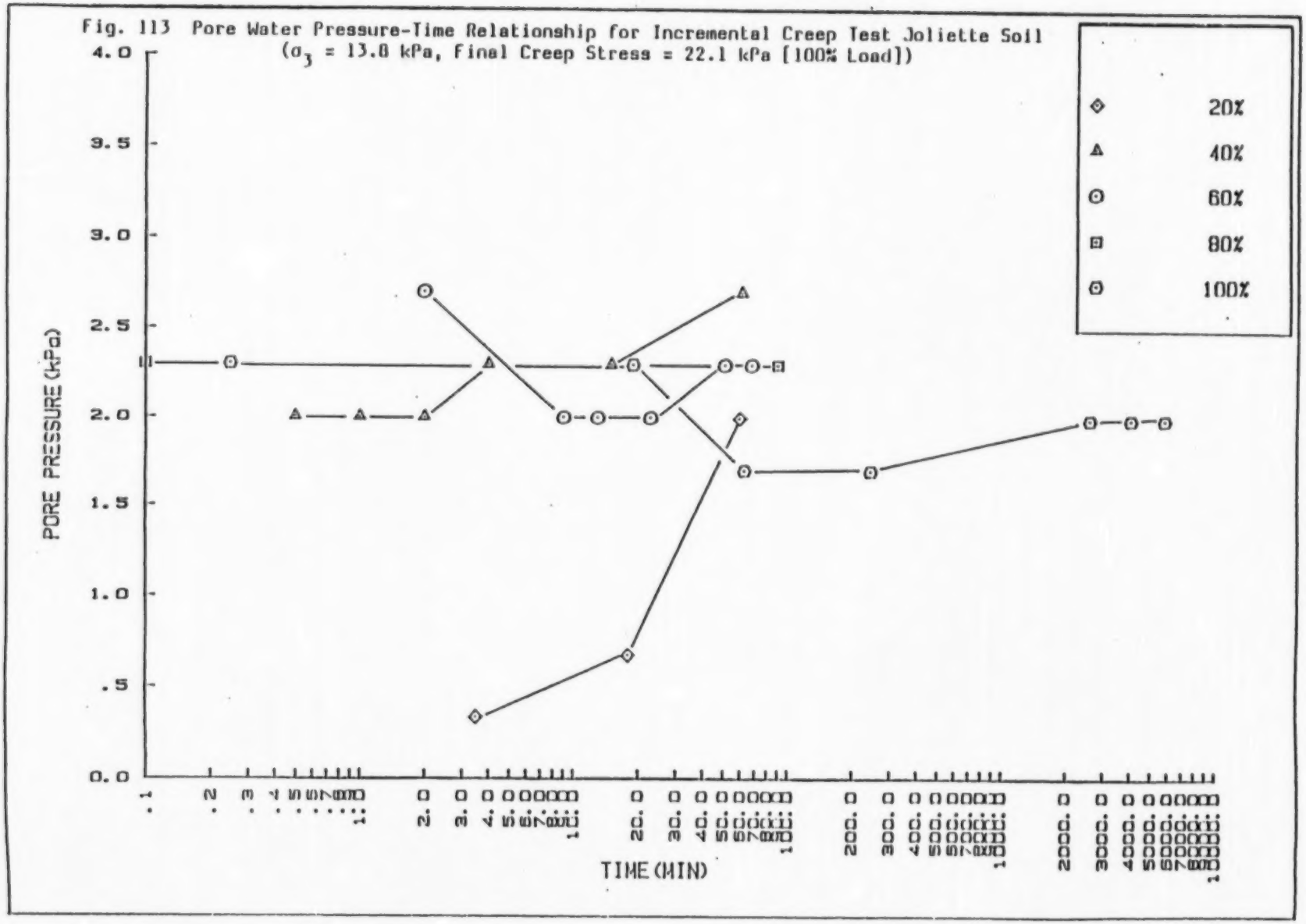


Fig. 114 Cumulative Strain-Time Relationship for Incremental Creep Test Joliette Soil (σ_3 = 27.6 kPa, Final Creep Stress = 27.6 kPa [100% Load])

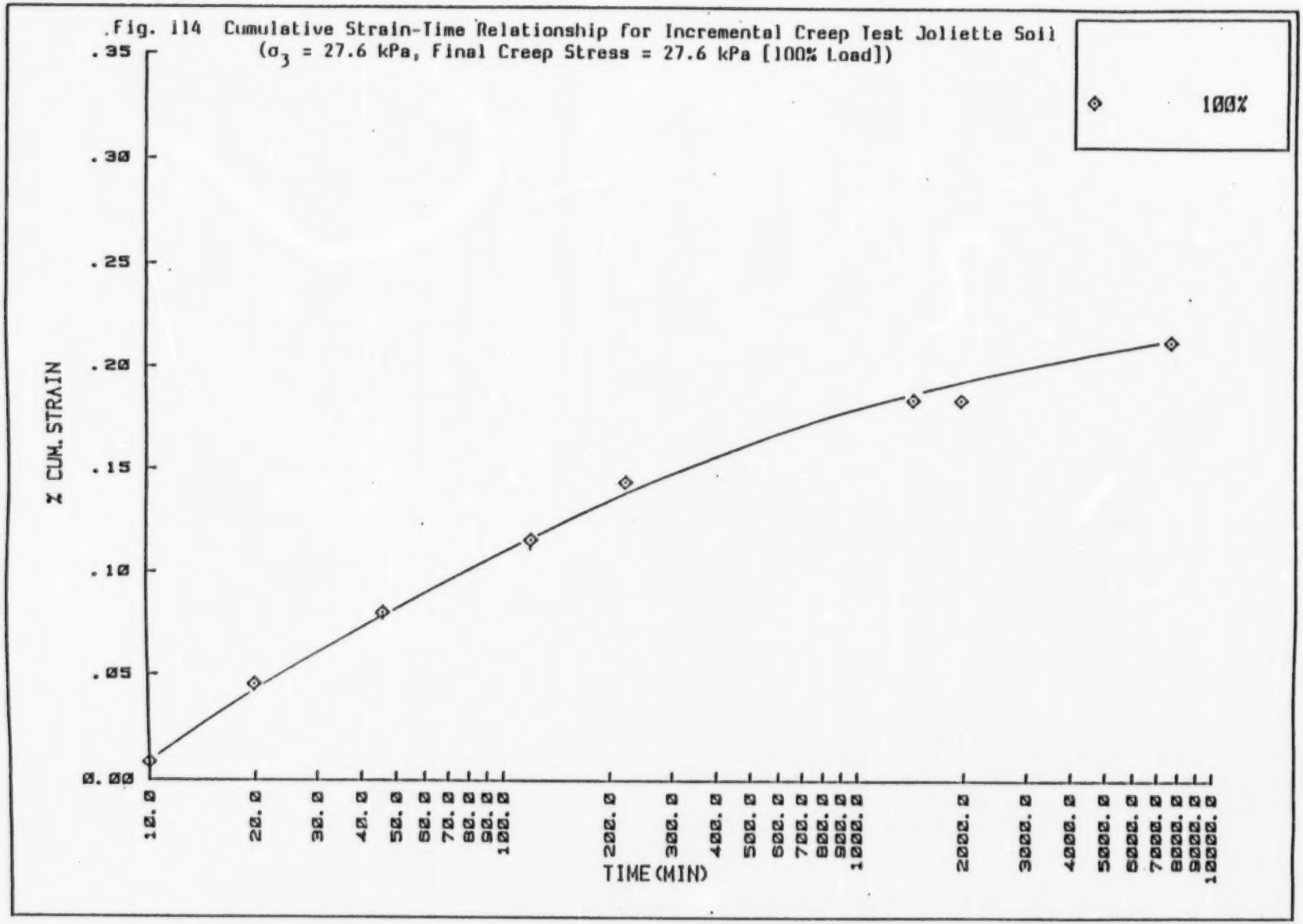


Fig. 115 Pore Water Pressure-Time Relationship for Incremental Creep Test Joliette Soil
(σ_3 = 27.6 kPa, Final Creep Stress = 27.6 kPa [100% Load])

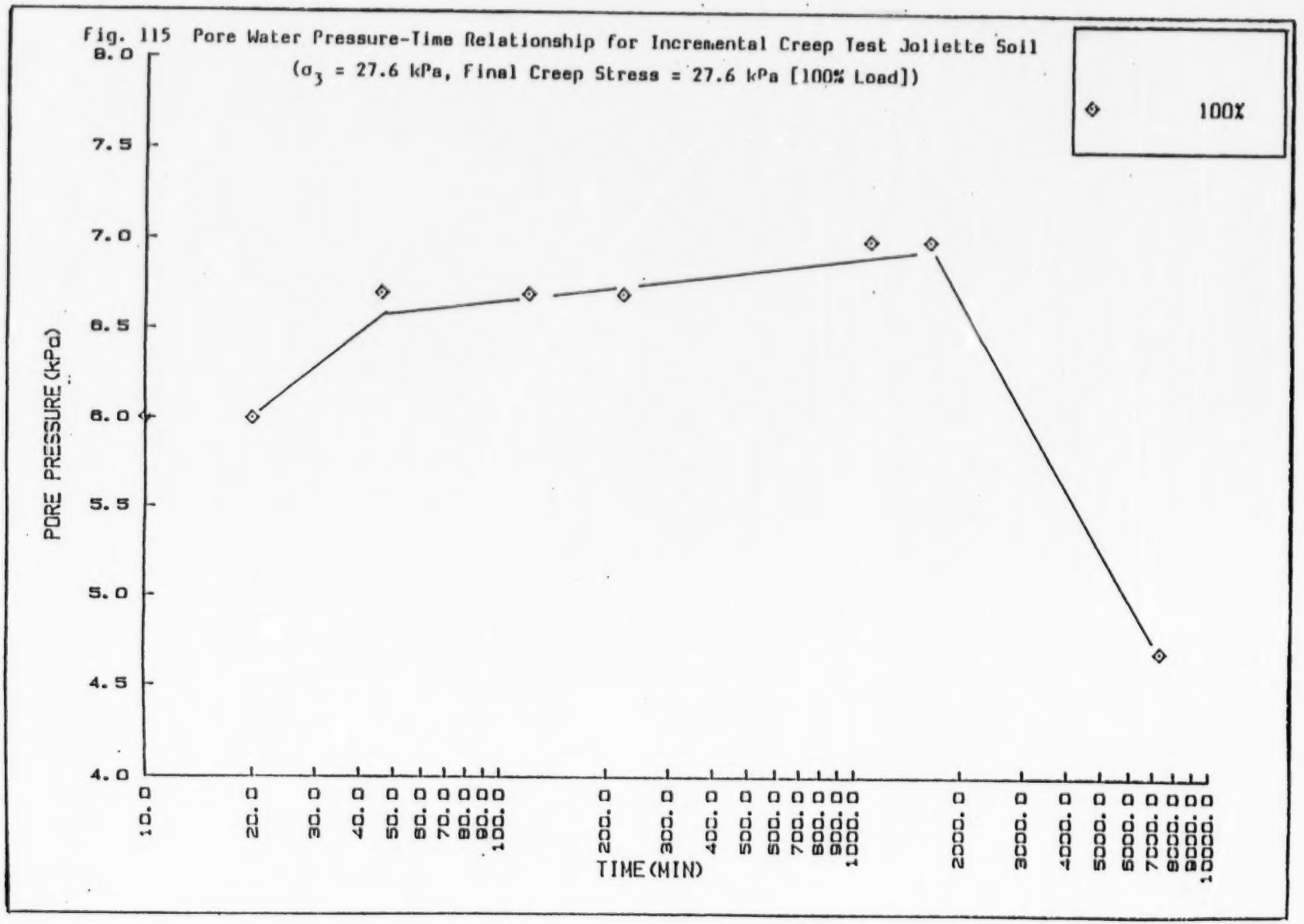


Fig. 116 Cumulative Strain-Time Relationship for Incremental Creep Test Joliette Soil
(σ_3 = 27.6 kPa, Final Creep Stress = 33.1 kPa [100% Load])

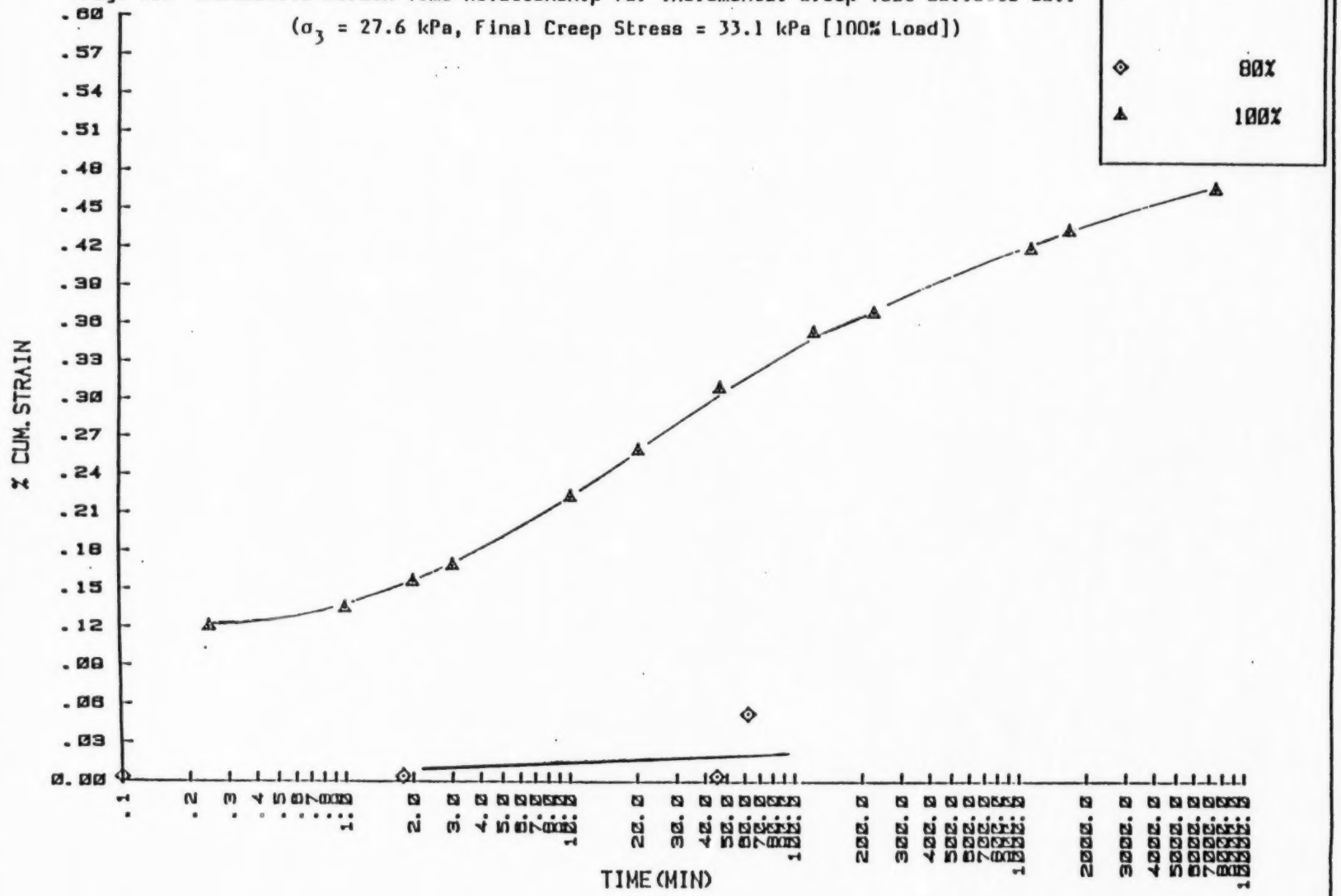


Fig. 117 Pore Water Pressure-Time Relationship for Incremental Creep Test Joliette Soil
(σ_3 = 27.6 kPa, Final Creep Stress = 33.1 kPa [100% Load])

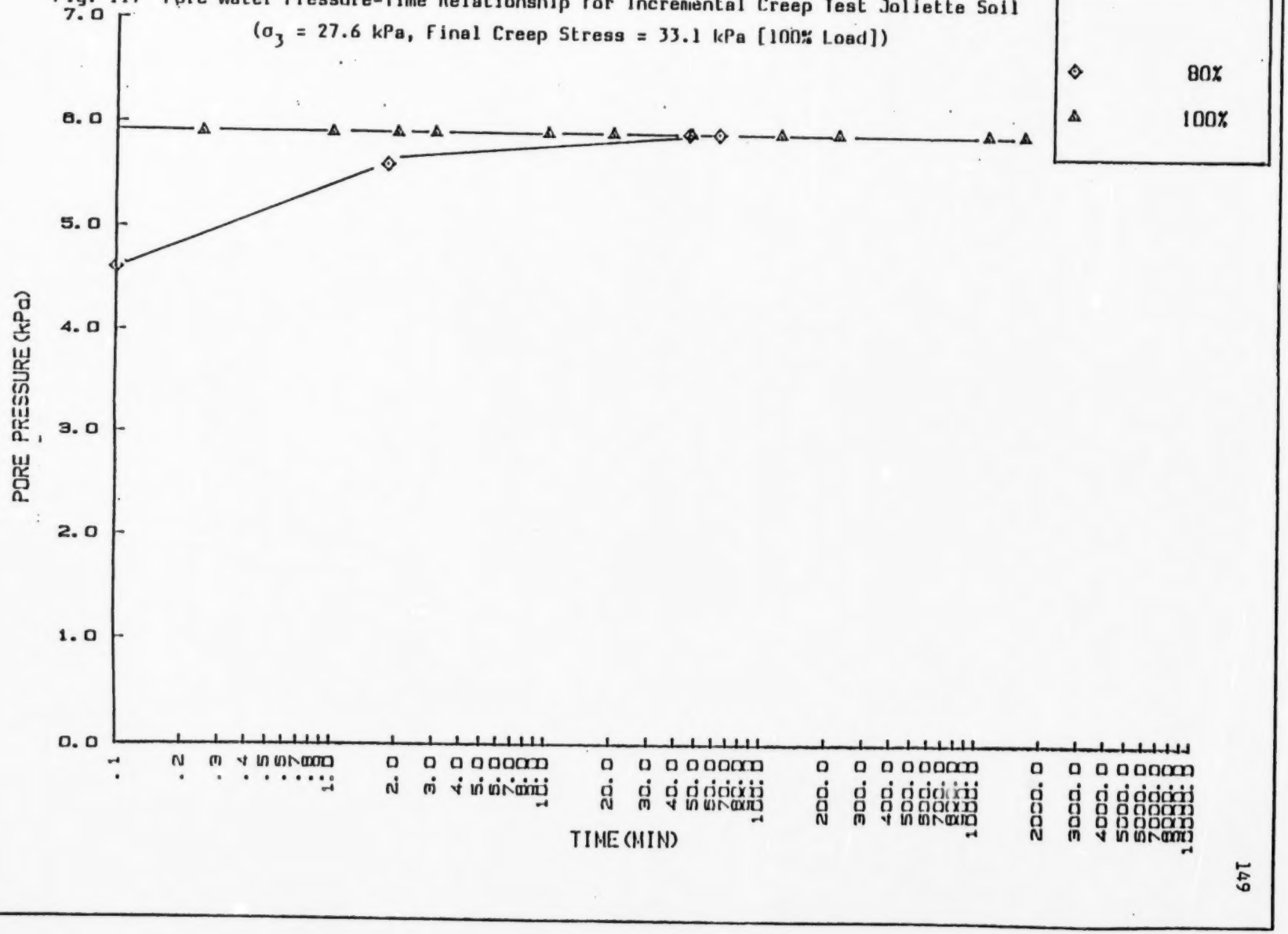
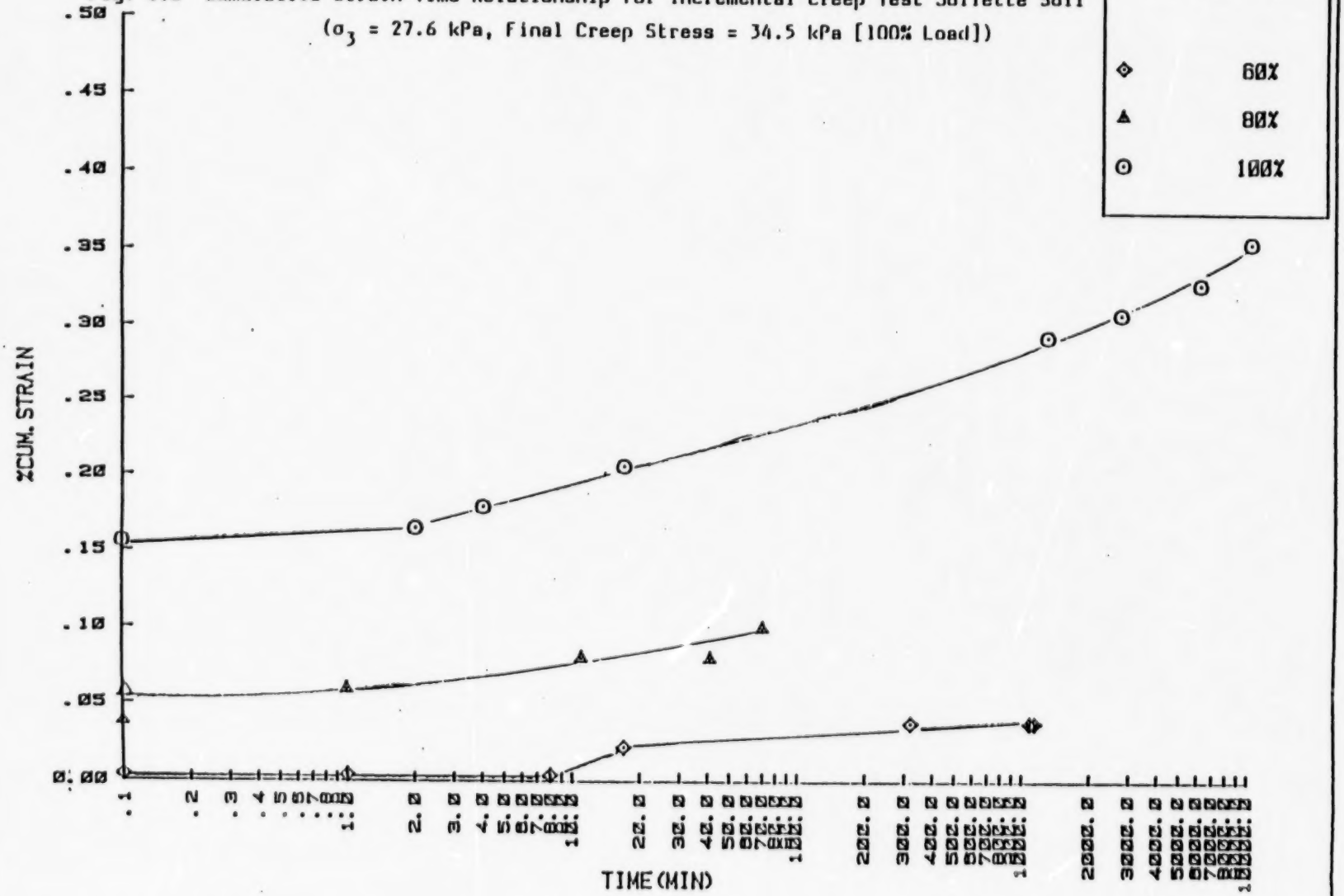


Fig. 118 Cumulative Strain-Time Relationship for Incremental Creep Test Joliette Soil
(σ_3 = 27.6 kPa, Final Creep Stress = 34.5 kPa [100% Load])



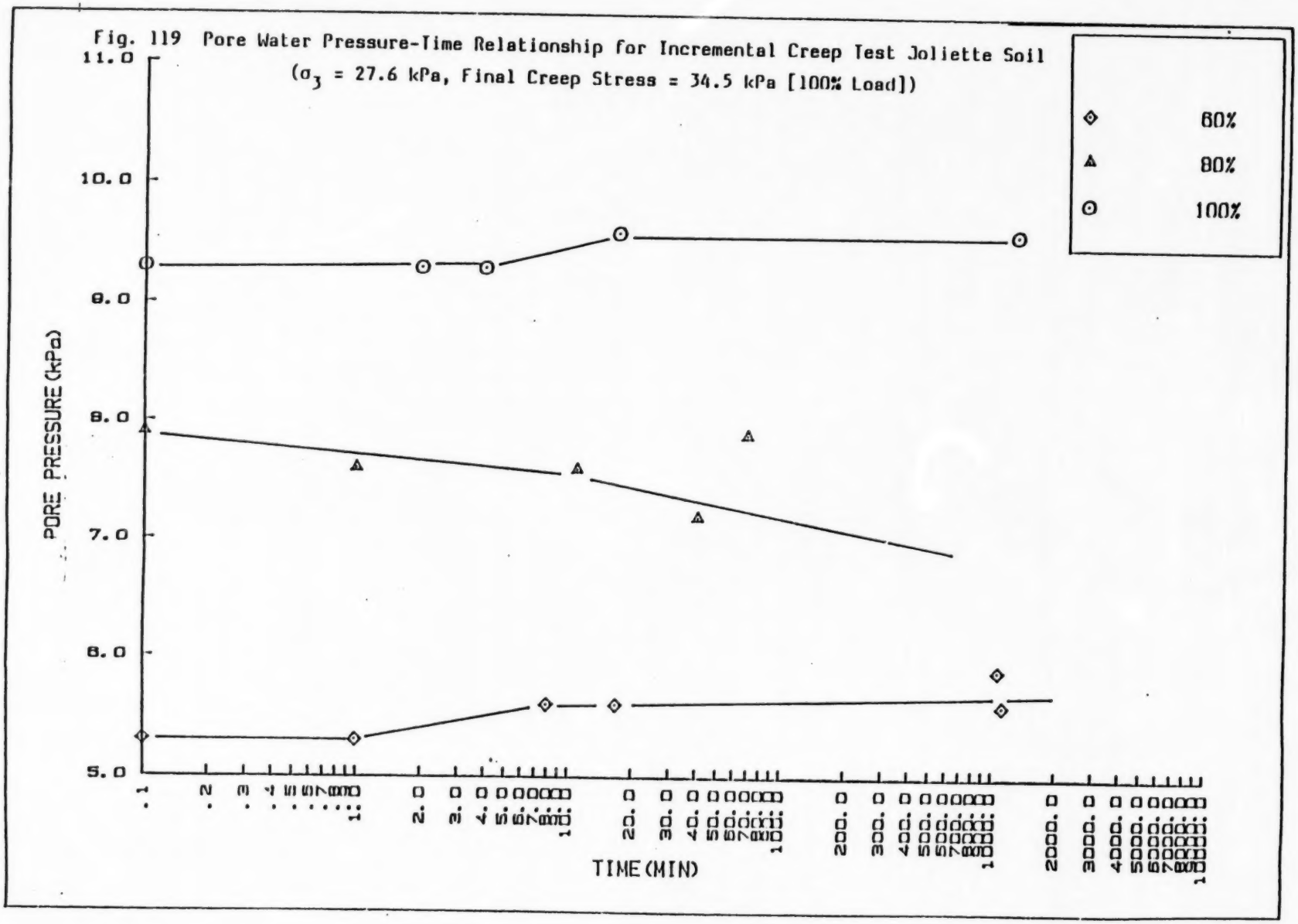


Fig. 119 Pore Water Pressure-Time Relationship for Incremental Creep Test Joliette Soil
(σ_3 = 27.6 kPa, Final Creep Stress = 34.5 kPa [100% Load])
60%
80%
100%
PORE PRESSURE (kPa)
11.0
10.0
9.0
8.0
7.0
6.0
5.0
TIME (MIN)

BIBLIOGRAPHY

Listing of Reports Published to date on CN Research Railroad Track Granular Depth Requirement Study.

1. "Predicting Requirements for Subballast Thickness, Progress Report No. 1, Measurements of Track Loads and Deflections, Edmonton, Alberta, May 1979"
   Transport Canada Report No. TP 2584
   CN Rail Research Report No. 158
   C.J. Dalton, J.F. Scott February 1980.

2. "Subballast Requirement Study, Dynamic Triaxial Testing Program, Present Status"
   Transport Canada Report No. TP6502E
   CN Rail Research Report No. 183
   Dr. D.E. Sheeran, Geotechnical Research Centre, McGill University (This report was completed July, 1979, but was formally issued under date of April, 1985).

3. "Prediction of Long Term Track Subgrade Settlement Due to Cyclic Wheel Loading"
   Transport Canada Report No. TP2648
   CN Rail Research Report No. 164
   Dr. D.E. Sheeran, H.H. Elmamlouk, E.A. Fattah Geotechnical Research Centre, McGill University, November 1980.

4. "Predicting Requirements for Subballast Thickness, Progress Report No. 2, Measurements of Track Loads and Deflections, Edmonton, Alberta, June 1980, L'Assomption, Quebec, August 1980"
   Transport Canada Report No. TP2987
   CN Rail Research Report No. 166
   C.J. Dalton, J.F. Scott April 1981

5. "An Interim Report for the Design of Ballast Thickness Under Adverse Conditions"
   Transport Canada Report No. TP3766
   CN Rail Research Report No. 177
   Dr. D.E. Sheeran, R. Joyaraman November 1980

6. "Predicting Requirements for Subballast Thickness, Progress Report No. 3, Measurements of Railroad Track Loads, Roadbed Dynamic Deflections, and Settlements, Period 1980 - 1982"
   Transport Canada Report No. TP4350E
   CN Rail Research Report No. 179
   C.J. Dalton, J.F. Scott March 1983